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FWS/OBS-81/48
September 1981

PROCEEDINGS: WORKSHOP ON DOWNSTREAM RIVER CHANNEL CHANGES RESULTING FROM DIVERSIONS OR RESERVOIR CONSTRUCTION

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FWS/OBS-81/48
September 1981

PROCEEDINGS
WORKSHOP ON DOWNSTREAM RIVER CHANNEL CHANGES RESULTING FROM DIVERSIONS OR RESERVOIR CONSTRUCTION

A Compilation of Papers, Summaries, and Problem Sets
Presented at Fort Collins, Colorado
27-29 August 1980

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DISCLAIMER

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PREFACE

The purpose of the workshop was to obtain opinions on the state-of-the-art in determining downstream changes in river channels which result from water diversions and reservoir construction. For this workshop three problems were selected and data assembled for each. Each participant then used the supplied data to determine, within the limits of the data supplied and the resources available, the nature of the change in the river channel which would result from some theoretical reservoir construction upstream. Two factors should be noted when reading the workshop papers:

1. the data supplied was a sub-set of the total data available, and

2. the participants had very limited resources to do their analysis.

Also the state-of-the-art of mathematical modeling using computer programs was not explicitly considered in the results of the workshop. For these reasons, the results of the workshop should not be applied without modification to any water management concerns about the real rivers used for these data sets; in other words, a prudent individual working on any one of these three rivers would use the papers in this proceeding as a very useful guide but do their own data collection and analysis. The appendix material included is representative of the data supplied; the purpose of including these data is to allow readers to read the papers and then use the appendix data in formulating their own approach to estimating the changes in the river channel.

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DOWNSTREAM RIVER CHANNEL CHANGES FROM DIVERSIONS OR RESERVOIR CONSTRUCTION

INTRODUCTION

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BACKGROUND

River systems are an integral part of the fluvial ecosystem. Streamflows, sediment transport rates, and channel morphology reflect the major responses resulting from river utilization activities. Knowledge of river mechanics, geomorphology, and watershed management is essential for formulating and selecting design alternatives by planners or engineers or both. The basic principles affecting the dynamics and response of streams to natural conditions or manmade alterations, or both, are not generally covered in college curricula, particularly at the undergraduate level. An understanding of stream mechanics is necessary for the proper planning and design of any channel change. This is particularly true of work in natural streams that carry heavy sediment loads. Analysis using principles of stream mechanics, that is a dynamic approach as compared to a static, rigid boundary approach, provides a more realistic understanding of channel response to man-induced changes. In response to this need, the U.S. Fish and Wildlife Service organized a workshop on the response of river systems to major impoundments or flow diversions.

This workshop assembled 20 of the world's most qualified individuals to discuss solutions to downstream problems stemming from hypothetical impoundments or diversions, or both, on three selected Western United States rivers. The participants were chosen from varied disciplines including geology, geomorphology, river mechanics, erosion and sedimentation, hydraulics, hydrology, fisheries and aquatic biology, and water quality. Each of the participants was an expert in one of these areas and possessed some understanding of all topics to facilitate a productive exchange of ideas.

Results of this workshop include this set of proceedings composed of reports prepared by the 20 participants along with reporters' comments and workshop summaries. These reports describe how each author perceives the problem of river response and determines a solution to a set of hypothetical man-induced changes. Reports were submitted before the workshop, reviewed within the group of workshop participants, and revised and resubmitted by the authors after the workshop.

The workshop offered a unique opportunity for a group of highly qualified people to present, exchange and discuss concepts, theories, and methodologies regarding the response of river systems to various types of development. The primary objective of the workshop was this set of papers which covers a wide spectrum of viewpoints and was prepared by experts in the area of river response and physical channel changes that result from man's activity.

Complexities and differing procedures involved in analyzing river response to works of man are often beyond the typical professional's expertise to assimilate into a usable form. This task is particularly difficult if the professional needing the information is not trained in the discipline that supports solutions. This is the situation, for example, when aquatic biologists investigating habitat dynamics require techniques for estimating channel changes caused by altered hydrologic and hydraulic processes. The workshop project was designed to provide information to the Water Resources Analysis Project (WRAP) from a large group of experts through a set of workshop proceedings.

WORKSHOP PHASE

River systems selected for analysis and discussion were Poplar Creek, a gravel and cobble stream in California, the Yampa River in Colorado, and the Elk River in Eastern Kansas. Participants were divided into three groups and requested to assess the importance of hypothetical development on one of the three river systems. After receipt of a data package containing information that might typically result from an initial reconnaissance and literature search, participants were requested to:

- Consider response of the river system to hypothetical development and alteration
- Present concepts, methods, and other aspects in a general way that you utilize and recommend for analysis of such channel problems
- Submit a paper including results of your analysis

Participants assembled at Colorado State University, Fort Collins, Colorado on 27 August, 1980 for the first of 3 days of workshop meetings. The first half day was devoted to an introduction by D. B. Simons and general discussion of workshop objectives and organization. C. B. Stalnaker provided a review of ecological and biological concerns in relation to instream habitat. R. T. Milhous supplemented the data provided to participants with a photographic reconnaissance of all three river study areas.

The next 3 half-day periods were devoted to presentation of papers by each of the participants for the Yampa River problem, Poplar Creek, and Elk River, respectively. A monitor for each group (K. Bovee--Yampa River, R. T. Milhous--Poplar Creek, and C. Thorne--Elk River) was responsible for the time schedule and organization of the presentations. During the final day the three groups met, separately, to discuss the results of the analysis of their particular problem area. A reporter for each group (C. Nordin--Yampa River, S. Schumm--Poplar Creek, and F. Theurer--Elk River) was assigned to insure that unanswered questions were addressed and to mediate unresolved issues. The reporter was assigned to identify key issues, and prepare a summary of problem perception and solution approach employed by the participants in each group.

The workshop concluded with a joint session to consider the reporter's summaries and complete any unfinished discussion. This final session concentrated on research needs in the area of river response and a discussion of possible alternative approaches to the problem of making the basic techniques of analysis of river response available to professionals not trained specifically in river mechanics or river engineering.

In organizing these proceedings an attempt has been made to adhere to the format of the workshop and, insofar as possible, convey the dynamics and "flavor" of the sessions. A brief statement of the workshop problems as posed to the participants is followed by a general summary of the results of the proceedings. Papers, revised and resubmitted after the workshop, are presented in the order discussed: Yampa River, Poplar Creek, and Elk River. Each set of papers is preceded by the reporter's summary presented at the final workshop session. Finally, a set of appendices contains detailed information drawn from the data packages provided to participants for each of the three rivers analyzed.

THE PROBLEMS[1]

YAMPA RIVER

The Yampa River is located in northwestern Colorado. The river is essentially a gravel-cobble bed stream above the confluence with the Little Snake River. Below the confluence the river is predominantly a sand-bed stream. Several reservoirs are proposed to be constructed on the Yampa River; one is the proposed Cross Mountain Reservoir and another is the proposed Juniper Reservoir. These reservoirs will impose changes in the flow and sediment regime of the river which will, in turn, cause possible alterations in the channel morphology.

The objective of this workshop problem was to discuss the short and long term changes of the following stream characteristics of the Yampa River as a result of the two reservoirs as follows:

1. Meander pattern
2. Configuration of the channel,
3. Substrate material, and
4. Pool-riffle sequence.

Data supplied by the U.S. Fish and Wildlife Service include information on four small reaches of the river, but only two are to be considered in any detail: the Box Elder reach and the Lily Park reach. Lily Park reach is located just above the Little Snake River, and Box Elder reach is located just above the juncton of the Yampa River with the Green River.

POPLAR CREEK

Poplar Creek (not the stream's true name) is a sand and gravel bed stream located in northern California. It is a tributary to the Sacramento River. Precipitation ranges from 70 inches in the headwaters to between 25 and 30 inches in the reach of interest below the proposed Dutch Gulch dam site. For purposes of discussion the reach of concern can be divided into two reaches as follows: Reach 1 between the dam and the junction of a major tributary, Dry Creek; and Reach 2 below the junction of Dry Creek. Participants were asked: "What will happen to the morphology of the stream's channel as a result of the changes in streamflows and sediment discharge caused by the construction of a reservoir upstream of a reach of stream?" Specifically, for both a short and a long time after construction of the reservoir:

---

[1] For purposes of the Workshop the physical characteristics of the three river systems were simplified and altered to a degree. Because of this "academic license," the results of the Workshop should not be interpreted as representative of the response to be anticipated if the proposed development were to be implemented.

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material? and
4. What will be the pool-riffle sequence?

ELK RIVER

Elk River is a tributary of the Verdigris River in the Arkansas River basin in southeastern Kansas. The Elk River reach selected for evaluation is near Longton, Kansas. Participants were asked to analyze and evaluate expected response of this reach of river to the installation of 45 small floodwater retarding dams in the upstream reaches of the watershed under the U.S. Department of Agricultural Soil Conservation Service small watershed program. Again, the question relative to the Elk River was "What will happen to the morphology of the stream channel as a result of the changes in streamflows and sediment discharge caused by the construction of a number of small flood retention reservoirs upstream of a reach of stream?" Specifically, for both a short and a long time after construction of the reservoirs:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material? and
4. What will be the pool-riffle sequence?

Acknowledgements: The assistance of the following individuals in providing data for the Workshop problems is gratefully acknowledged.

Yampa River: T. D. Steele & D. Bauer
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Poplar Creek: M. Gee
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R. L. Hager, State Biologist
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SUMMARY

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INTRODUCTION

Alluvial river systems such as the three examined in this workshop are very dynamic in nature and generally experience significant changes in depth, width, alignment, and stability with time. A systematic analysis is required to distinguish between changes due to the natural dynamic characteristics of the system and those due to man's activities. The changes may be defined as degradation, aggradation, and lateral migration. Degradation and lateral migration can endanger adjacent property, bridges, and other hydraulic structures while aggradation can reduce channel capacity, increase lateral erosion, and increase the flooding potential.

The dynamic nature of river and watershed systems requires that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in a river frequently initiate responses that can be propagated for long distances both upstream and downstream (Simons and Senturk 1977). Successful river utilization and water resources development require a general knowledge of the entire watershed and river system and the processes affecting it. This goal can be achieved only through a basic understanding and application of physical processes governing channel response and the utilization of physical and numerical techniques.

In the past the emphasis of research and analysis has been on rivers with fine-grained alluvial beds and it is only recently that attention has been focused on flow in gravel-bed channels such as the Yampa River and Poplar Creek. In recent decades, though, increasing human involvement with upland and mountain regions, activities such as agriculture, forestry, recreation, gravel mining, reservoir construction, river regulation, and highway construction have affected the gravel-bed river environment. Gravel-bed rivers have therefore increasingly felt the impact of human activities and have themselves become the focus of engineering projects. As a result, there is an urgent need for the development of dynamic modeling techniques that can be applied to the management of gravel-bed rivers.

This summary is intended to highlight the major themes of the participants' papers regarding the scope, approaches, and data requirements for analyzing the general question of downstream river channel changes associated with diversions or reservoir construction. The summary will also relate these major themes to the instream aquatic habitat, a task not specifically assigned to the participants in their analysis. A careful reading of the papers which follow will reveal striking consistencies in these areas as well as several interesting digressions from what can be considered the usual thrust of river system analysis. One should keep in mind when reading these papers that given the techniques currently available for analysis of river channel change, the "science" of river engineering must be supplemented by subjective judgements based on years of field experience with rivers. The papers collected in these proceedings provide an exposure to the "art" of river system analysis that is not normally found in the technical literature.

MAJOR PROBLEMS THAT REQUIRE DETAILED ANALYSIS

The major effects imposed by diversions or reservoir construction that must be evaluated are as follows:

1. Determine conditions on the watershed such as climatology, hydrology, land use, possible land use changes, soil types, the geometry and topography of the system, and the existence of man-made or natural controls or both.

2. Determine the characteristics of the proposed reservoir including its volume, geometry, its stage-volume and stage-area curves, the operational plan for the reservoir, its trapping efficiency, and its uses such as irrigation, power, or recreation.

3. Evaluate the impact of the reservoir on flow in the study reach. The type of hydrographs normally experienced will depend on operation of the reservoir, probably reducing peak flows and increasing base or minimum flows

4. The water released from the reservoir will be transporting less sediment at the point of release than natural flows and this change in water quality may induce degradation, bank erosion, head cutting in tributaries, and possibly may induce growth of aquatic plants that may effect flow conditions, water losses, and water quality.

5. Evaluate the impacts of changed flow conditions on river form, the sequence of riffles and pools, lateral migration, and the bed material.

6. The modification of flows will cause changes in channel regime such that the aquatic habitat of the river may be affected, at least until a new equilibrium is established.

7. The storage and modified release of water from the reservoir may cause changes in the natural temperature conditions in the reach below the dam.

8. The reduction in peak flows and base flows will cause changes in the hydraulic characteristics and possibly in the stream morphology that may alter the fish spawning environment.

9. Impacts of the reservoir on groundwater conditions near the reservoir may be significant and should be investigated.

In order to evaluate the above-mentioned effects, an analysis of hydrological, hydraulic, morphological, and thermal changes is required. Hydrologic analysis will establish the flow occurrence frequencies for all of the main rivers and major tributaries. Hydraulic analysis will estimate the hydraulic parameters such as velocity, depth, top width, and wetted perimeter that are required to conduct the sediment and morphological studies and evaluate changes induced in the fish spawning capacity of the system. Sedimentation analysis will analyze the impact of siltation on the hydraulic parameters that govern fish habitat due to construction of the project. Careful scheduling of construction activities may be required to minimize impacts. The morphological studies will consider the changes that can be expected to occur over time in the river profile and cross section along a study reach. A thermal study should consider the thermal regime in the river resulting from selected withdrawal of water from a multilevel intake structure.

PROPOSED SCOPE OF WORK

The following general scope of work is suggested by the participants' papers in order to adequately analyze the responses of a reach below a dam or reservoir.

1. Conduct site visits to become familiar with the physical environment. All participants agreed that this is absolutely essential to any adequate analysis of the system.

2. Collect, collate, synthesize, and verify available hydrologic, hydraulic, thermal, topographic, sediment, cross-sectional, geological, structural, and fish habitat data pertinent to the study (see the data base section which follows).

3. Evaluate the data available for analysis, identify the immediate data gaps, and recommend an effective short term in-field data collection program.

4. Conduct in-field data collection of cross-sectional data, velocity, depth, width, and bed material and suspended sediment samples in the river system of concern. Participants agreed unanimously that the data packages provided were not adequate for more than a preliminary qualitative assessment and must be supplemented to support quantititive analysis.

5. Compile and develop a spatial representation system that approximates the study area. This spatial design will provide a line diagram showing the watersheds, reservoir, river mile, cross-sectional numbers, location of structures, fish habitat reaches, bed material sampling points, geologic controls, and works of man such as construction roads.

6. Review and evaluate the hydrologic changes in the river system induced by the dam.

7. Estimate sediment loading and the associated impacts on the study reach during the construction phase (this requires site data).

8. Estimate sediment loading to the river system during the construction phase.

9. Establish the resistance to flow equations and sediment transport characteristics for the study reach.

10. Conduct a qualitative morphological analysis (degradation, aggradation, planform change, and bank stability) of the system considering the clear water release from the dam. The changes in flow over the long term future should be assessed. The analysis would provide information on the expected bed profile and cross section and bed material distribution over time.

11. Evaluate the changes in the hydraulic parameters that affect the fish habitat utilizing a water-sediment routing program.

12. Conduct an initial thermal study of the temperature regime in the study reach and the impacts of the reservoir. If changes in thermal regime are found to be significant, a more detailed study will be necessary that considers the thermal routing in the system and selected withdrawal of water from multilevel intake structure from the reservoir to improve the thermal regime. This requires a mathematical model study.

13. Use a mathematical model if a more detailed study of the thermal regime is required to identify the potential thermal problem associated with fisheries. Suggestions to modify the position and openings of the multilevel intake should be made if the study shows that serious temperature effects occur.

14. Evaluate one proposed construction plan and, if required, recommend alternatives for evaluation.

15. Prepare reports documenting the results of analysis and recommendations.

PROPOSED APPROACH

Storage and release of water from a reservoir will have effects on instream flow, discharge rates, channel morphology, velocity, substrate, depth, top width, and temperature. The modification of flow in a reach below a reservoir also may have both beneficial and adverse effects on the fisheries over time. The hydraulic, morphological, and thermal changes, as well as channel stability, are functions of hydrologic changes. A systematic approach to the analysis of the hydrologic, hydraulic, sedimentation, morphological, and thermal changes is required to adequately evaluate the potential effects on the creek and its fish habitat.

The sediment transport capacity can be estimated by applying the modified Meyer-Peter, Muller equation using limited measured data. The sediment loading to the stream due to construction activities can be estimated by applying an on-site soil erosion model (Simons et al. 1977).

The morphological changes in the system are dynamic in nature and should be evaluated in both qualitative and quantitative terms. The altered water and reduced sediment flows downstream of the reservoir will probably induce degradation or aggradation depending on the sediment retention capacity of the reservoir. The altered flow and the clear water release from the reservoir will probably degrade the downstream channel; however, the extent of degradation is dependent on the flow rate, particle size, channel shape, and downstream controls.

The degradation or aggradation of the river system, or both, can be estimated by applying an acceptable water and sediment routing method such as the one developed by Li and Simons (1979). This method routes sediment by size fraction and has been verified in many field applications. The new equilibrium morphological conditions can be further checked by utilizing the Shields criteria (Simons and Senturk 1977) considering the armoring effect. After the morphological changes have been evaluated, the hydraulic parameters related to fisheries such as top width, velocity, flow depth, and substrate can be determined by applying an acceptable model.

An initial thermal study should be conducted. A more detailed analysis must be performed if significant changes in the thermal regime are detected by the initial study. This analysis would consider the thermal routing in the system. A mathematical model would be required to conduct this detailed analysis. A candidate for use is the Colonell (1976) model, which was developed by modifying the Massachusetts Institute of Technology model (MIT Model). The model was based on the simultaneous solution of appropriate equations for the conservation of mass and energy. Related hydrodynamic and thermodynamic processes include 1) internal radiation absorption, 2) heat sources and sinks, 3) advective heat transport, and 4) convection and diffusion. In order to account for cold region conditions, the model can be modified to consider ice cover during the winter months.

The general approach to the complicated problems involved in the study of downstream river channel change is 1) to consider the significance of the physical and biological environment, 2) to conduct sensitivity analysis for evaluating the relative importance of the physical processes and data, 3) to adhere to the project schedule, and 4) to provide a factual, practical, efficient, and effective solution.

DATA BASE

The data required to conduct the required hydrologic, hydraulic, sedimentation, and morphological analysis generally will include:

A. Watershed:

Geometry
Topography
Road location
Snowmelt rate
Soil type
Geology
Vegetation

B. Surface Water Hydrology:

Discharge records
Stage records
Stage-discharge relationships
Flood frequency curves
Flow duration curves
Design flood hydrograph
Sediment transport data (if available)
Tidal waves

C. Cross-sectional Data:

Location map
HEC-2 cross-sectional data covering the study area

D. Bed and Bank Material:

Size
Size distribution
Banks (stratified or homogeneous)

E. Structural Data:

Dam
Hydroelectric facility
Bridge
Construction plans

F. Geological Control Data:

Rock outcropping
Narrow section
Manmade control

The additional data requirements for a thermal study include:

Incoming solar radiation
Atmospheric radiation
Air temperature
Relative humidity
Wind speed
Water surface elevation
Upstream inflow rates to the reservoir and temperature

Outflow rates of the reservoir
Geometry of the reservoir
Water transparency to solar radiation
Water temperature
Time and periods of breaking and forming of the ice cover

As with the workshop data packages, the available data will seldom satisfy the requirements of this data list. However, appropriate methods of synthesis can be used to supply necessary data from a secondary data base. In general, these data can be classified as 1) data essential to conduct the proposed study, 2) data that are available on existing records, 3) data that can be synthesized by theory or extrapolated from adjacent basins, and 4) data that must be collected to supplement the existing data and proposed synthesized data to add to the validity and accuracy of the study.

CONCLUSIONS

There was general agreement among the workshop participants concerning the critical problems and data needs for assessing downstream channel change. The scope of the analyses provided in the papers which follow was influenced by the time available for study, the data provided, and the absence of a site visit to the study area. As a result participants generally attempted only a qualitative analysis, but many papers outline procedures for more detailed quantitative studies.

The details of the approaches selected by each participant vary based on the individual's training and experience. There are, however, striking commonalities which highlight the state-of-the-art of river system analysis.

Regime theory and concepts of dynamic equilibrium are used widely as a basis for qualitative assessment of river response to development. The concepts of fluvial geomorphology derived from Lane and Schumm are also pervasive. All participants relied heavily on experience, and many supplemented their analysis with case study data derived from comparable physiographic settings. It was generally agreed that hydraulic engineering projects can induce major changes in the hydrologic, hydraulic, and sediment regimes of river systems, and at present, theory alone is not capable of predicting this complex response. Research effort committed to producing documented case studies can provide an important resource for evaluating river response. Post-project monitoring and analysis should be supported by agencies responsible for river system development and control.

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REPORTS ON YAMPA RIVER WORKSHOP PROBLEM

YAMPA RIVER

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PROBABLE IMPACT OF PROPOSED RESERVOIRS ON SEVERAL REACHES OF THE YAMPA RIVER IN COLORADO: SUMMARY OF DISCUSSIONS

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BACKGROUND

Two reservoirs are proposed by the Colorado River Water Conservation District for construction on the Yampa River in Colorado. According to information received from the District's Public Affairs Consultant (Lee Harris, personal communication, 1980), the upstream reservoir, Juniper, will have a capacity of about 1.08 x 10^6 acre-feet behind a 210-foot dam, at water surface elevation of 6125 feet. It will have a total rated generating capacity of 98 x 10^3 kw and will generate 134 x 10^6 kw-hrs/year under a schedule for peaking power. Juniper Dam will have a minimum release of 25 cfs (cubic feet per second) and the capability of going to a maximum discharge of 7000 cfs instantaneously.

The downstream reservoir at Cross Mountain is a re-regulating reservoir to smooth the flows from the peaking operaton at Juniper. It will have a capacity of about 142,000 acre-feet at water surface elevation of 5875 feet behind a 260-foot dam and will have generating capacity of 50 x 10^3 kw. It will generate about 165 x 10^6 kw-hrs/year. Maximum outflow is about 3000 cfs. Proposed operating criteria call for a minimum release of 200 cfs or a greater amount to provide a minimum flow in the Yampa River below its confluence with the Little Snake River of 500 cfs. During the rafting season, May through July, a minimum flow of 1800 cfs will be maintained. All the above data are preliminary and are subject to revision.

Of concern here is the question of what will happen to the morphology of the stream channel as a result of changes in streamflow and sediment discharge brought about by these reservoirs. In particular, the following questions were posed.

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material, and
4. What will be the pool-riffle sequence?

Cross section data were provided for four reaches of the river.

1. Maybell
2. Lily Park
3. Mantle Ranch
4. Box Elder

Only two reaches, Lily Park and Box Elder, were to be considered in any detail. Additional information included excerpts of background material from Steel et al. (1978), a report by Andrews (1978) on sediment yield, monthly flow records from the gaging stations, Yampa River near Maybell and Little Snake River near Lily, for the period 1910-1976 and projected flows with the two reservoirs for both stations for the period 1927-1976. Peak flow records, copies of daily sediment loads, a couple of snowmelt hydrographs, and topographic maps were also furnished. Particle size distribution for only one sample of bed material was provided; it was collected near the Maybell gaging station.

APPROACH

Introductory sessions of the workshop included an overview by D. B. Simons, a review of ecological concerns with particular attention to in-channel habitat, by C. B. Stalnaker, and a summary of the characteristics of each river by R. T. Milhous.

After the preliminary discussions, R. M. Li presented a general review of the Yampa River problems with a summary of available data and a careful exposition of the three basic levels of analysis: 1) qualitative involving geomorphic concepts, 2) quantitative involving geomorphic concepts and basic engineering relationships, and 3) quantitative involving sophisticated mathematical modeling concepts. He presented some qualitative assessments and outlined steps leading to a few results from the second level approach. He concluded by emphasizing the importance of a field site investigation and of the need for more complete data before any detailed computational modeling could be undertaken.

Dr. Li's introduction was followed by the participants' presentations and informal discussions by the group to finalize concepts regarding the approach. The level of analysis by the participants varied from fairly complex computations to qualitative discussions of the general geomorphology and of the data needs both for preliminary assessment and for detailed evaluations.

A number of aerial photographs and slides of the critical reaches of the river were reviewed during the course of the workshop. Most of the participants plotted the river profile from topographic maps provided. Consideration of the profile, the topographic maps, slides and photographs, and the background information led to the following conclusions regarding the nature of the river.

1. The Yampa River is not an alluvial channel; its slope is controlled by the bedrock through which it is cutting its major canyons. Its pattern and behavior are controlled mostly by geology rather than by the quantity of water and sediment it is required to convey.

2. The channel has a pool-riffle configuration over much of the reach considered. Most of the riffles are formed by cobbles and boulders fed into the channel from steep, mostly ephemeral tributaries.

3. Some reaches of the river are alluvial in nature. The bed is armored with cobbles the size of which corresponds roughly to the size for critical tractive force at bankfull discharge.

4. These gravel-armored reaches are relatively stable.

5. Small quantities of sand and fine gravel are available for transport in the Yampa River above the Little Snake confluence; these sediments move over the armored gravel bed and accumulate in pools, on point bars, and in certain reaches with mild slopes.

6. Much of the sediment transported to the Box Elder reach is sand from the Little Snake River.

7. The reservoirs will trap most of the sediment of the Yampa River now passing the Maybell gage.

Much of the discussion centered around data needs. It was generally agreed that the data provided were not adequate. In particular, it was pointed out that meander pattern and channel configuration in the alluvial reaches of the river would likely depend on some dominant or "channel forming" discharge that might occur only a few days a year. The only information on reservoir releases were average monthly flows, whereas specific operating rules were needed, along with information on the outlet structures, their elevation, whether or not they would release sediments, and so on. There also was general concensus that extensive bed samples and at least some general information on the nature of the bank material were essential to any quantitative analysis.

One critical question emerged. What would happen in the vicinity and downstream of the confluence of the Little Snake and Yampa Rivers? The Little Snake River contributes heavy sediment loads. Would the regulated flows be able to transport this sediment, or would there be aggradation in the reach? Would the transport capacity of the sustained sediment-free reservoir releases exceed the supply of sediment, resulting in degradation? Only one of the participants, Dr. R. J. Garde, presented any calculations on transport capacities. He concluded that there would be minor degradation and armoring at Deerlodge Park, and that "once the surface is paved the material that comes in from the Little Snake River would be safely carried down the Yampa River." He was careful to point out that his analysis was based on some "simplified assumptions" and a careful reading of his paper will show that these simplifications were necessary because of the lack of data.

It was generally agreed that more detailed flow and transport calculations would be necessary to resolve the question of what would happen in the vicinity of the confluence of the Little Snake River, and that this would require the collection of field data to provide cross sections and bed material size distributions. In addition, it was stressed by Dr. Shen that future flows were stochastic processes; the distribution of these flows would have to be determined and hydraulic calculations should be carried out to cover that distribution. The future conditions in the reach cannot be predicted precisely; they should be specified in terms of their probabilities of occurrence.

Several other important concerns were identified and discussed by the group. These may be summarized as follows.

1. The method of analysis will depend to a large extent on whether or not the river system is stable, so a historical perspective is important. Any old maps, aerial photos, geological maps, or historical reports would be useful.

2. There is a potential in the basin for drastic land-use changes in connection with coal mining and other energy related industries. These could have appreciable impact on the flow and sediment discharge of the Little Snake River, so they need to be considered.

3. Channel configuration downstream of reservoirs is influenced in many circumstances by the encroachment of vegetation. Not much is known about this; it is an area in which additional research is needed.

4. The sites should be inspected by aerial reconnaissance and on the ground.

SUMMARY AND CONCLUSIONS

In summary, the participants of the workshop presented a variety of approaches, from qualitative geomorphic assessments to quantitative calculations. A number of important concerns were raised during the group's discussion; most of these revolved around the need for additional data. There was general agreement on the following three major points.

1. The question of whether or not future flows would be able to transport the sediment delivered by the Little Snake River is critical to the analysis.

2. To answer this question, some computations of the hydraulics and the sediment transport will have to be carried out.

3. Additional field data are needed to provide a basis for reliable computations, and in fact, additional data are needed even for a qualitative assessment of the impacts of reservoir construction on the system.

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YAMPA RIVER

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INTRODUCTION

Several small flood control reservoirs are to be constructed on the Yampa River, one at Cross Mountain and one at Juniper. These reservoirs will impose changes on the flow and sediment regime of the river which will, in turn, cause changes to the stream channel. More specifically, this study is concerned with short and long term changes to the following stream characteristics:

1. Meander pattern,
2. Channel configuration,
3. Substrate material, and
4. Pool-riffle sequence.

The reaches that are treated in detail are the Box Elder Reach and the Lily Park Reach which are located 2 miles and 52 miles from the confluence with Green River, respectively.

In order to assess the short and long term changes to various river characteristics, several levels of study should be undertaken. For the Yampa River, the levels of study proceed as follows.

Level 1. Preliminary appraisal of river processes.

Level 2. Assessment of the scope of the problem.

Level 3. Additional data requirements.

Level 4. Analysis of river processes and prediction of changes.

The forthcoming comments will treat each of the above levels.

PRELIMINARY APPRAISAL OF RIVER PROCESSES

A preliminary appraisal can only be undertaken if the following data are available.

Topographic maps

Geologic maps

Air photos

Site inspection or ground photos

River cross sections

Hydrologic information

For the Yampa River problem, topographic maps were supplied and air photos of the Lily Park Reach were obtained during the workshop. A general appraisal of the two reaches follows.

Box Elder Reach

The river channel may be described as having an irregular meander pattern, deeply entrenched in a 1000 foot deep canyon, with a thin layer of alluvium on the bed. The banks are essentially bedrock. There are no air photos, consequently it is difficult to assess the river processes, but it appears that the river is stable and is not expected to change its pattern.

Lily Park Reach

As noted from the topographic maps and the air photos, the Yampa River changes character several times as it emerges from the Cross Mountain Canyon; in fact this reach can be further subdivided into three distinct reaches as follows (see Figure 1 for details of reaches as traced from air photos).

Reach No. 1. The Yampa River is confined within stable steep banks, has a pool and riffle sequence with some midchannel bars composed of gravel and boulders. The channel pattern is irregular with some bends and straight reaches. It appears that the sediment load brought into this reach from the Cross Mountain Canyon would be transported through the Reach and that the channel is relatively stable (see Figure 2 for river profile).

Reach No. 2. The Yampa River becomes wider with a regular meander pattern. The banks consist of a high terrace on the left which appears stable and a low floodplain on the right which is susceptible to erosion. This reach has several point bars and could be aggrading due to the confluence at the bottom end of the reach.

Reach No. 3. The Yampa River widens significantly and flows within low unstable banks. The bed has large midchannel bars as well as wide point bars indicating that material is depositing throughout the reach and that the bed is aggrading.

With the various reaches described, we can now discuss the scope of the problem to be studied.

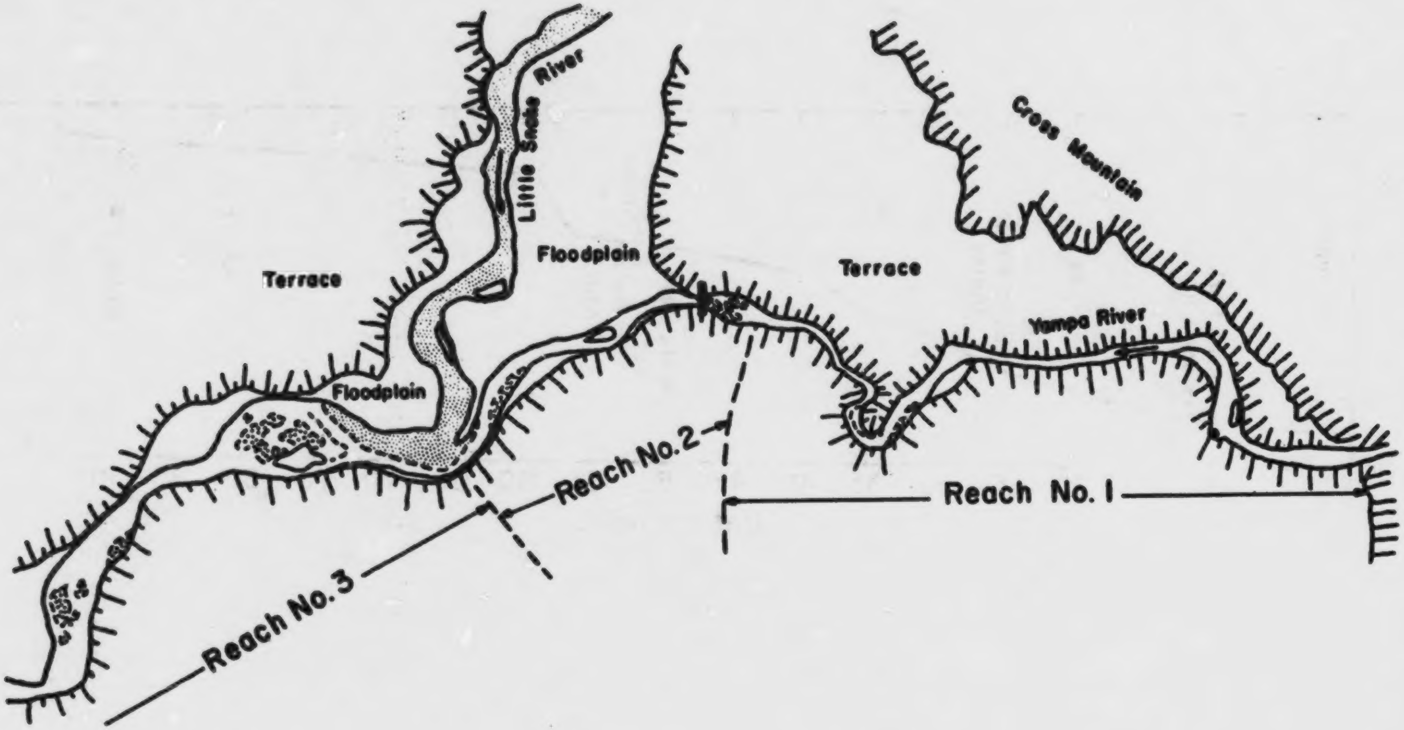

Figure 1. Yampa River at Lily Park.

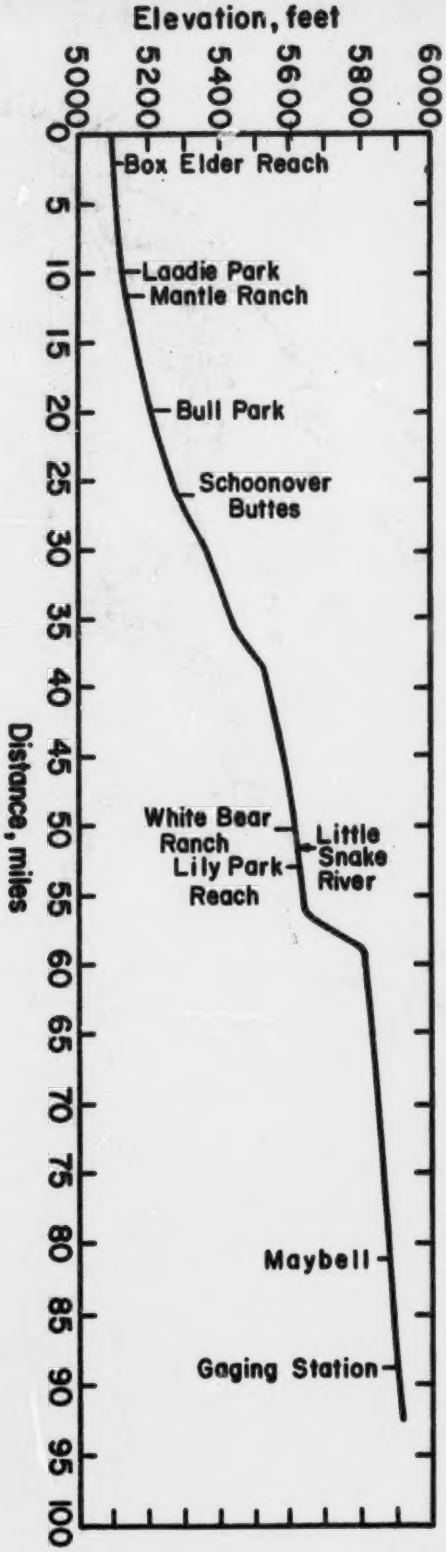

Figure 2. Yampa River profile.

ASSESSMENT OF THE SCOPE OF THE PROBLEM

Box Elder Reach

Examination of the cross sections in this reach, for three distinctly different discharges, shows no changes. This indicates that the bed and banks are primarily composed of resistant material and, further, that the stream characteristics of concern will not be significantly altered by the construction of upstream reservoirs. Since the channel is deeply entrenched in bedrock its configuration and its meander pattern should not change markedly, at least not within the engineering time scale. The pool and riffle sequence as well as substrate material will also not be affected since reservoir release flows will have smaller magnitudes than historical floods.

Therefore, there is probably no additional data or analysis required for this reach.

Lily Park Reach

In terms of fish spawning grounds, it is not certain which of the three distinct reaches are of most interest to biologists; therefore, all three reaches will be discussed briefly.

The problem consists of predicting the changes to meander pattern, channel configuration, substrate material and pool and riffle sequence. We can consider these changes for each reach.

	Reach No. 1	Reach No. 2	Reach No. 3
a) Meander Pattern	No change, stable	Some change	Minor changes possible
b) Channel Configuration	No change, bed-material stable	Some change, depends on backwater	Some change, depends on reservoir releases
c) Substrate	No change	Some change, depends on backwater	Some change, depends on reservoir releases
d) Pool & Riffle	No change	Some change	Uncertain

It appears that Reach No. 1 will not undergo major changes. A reservoir upstream will probably reduce flood peaks and the existing pool and riffle sequence will probably not be disrupted.

As for Reach No. 2, there may be some changes to the meander pattern and the channel configuration if there is significant bank erosion, which could be occurring along the right bank. In regard to substrate, pools, and riffles, there may be some infilling of the pools with sand and gravel depending upon the extent of backwater from the Little Snake River.

For Reach No. 3, significant changes could occur. The construction of dams on the Yampa River will result in reduced bed material transport capacity. Because the supply of bed load from the Little Snake will not change, the Yampa may aggrade at the confluence. Also, if the peak flows on the Yampa are reduced significantly, then upstream progressing degradation will occur on the Little Snake during corresponding high flows (Figure 3). Therefore, it is probable that the Yampa River will aggrade in Reach No. 3, this would create backwater effects up the Yampa and drown out the riffles. With this flow situation, sands and gravels moved into the pools will probably stay there, at least until high flows are released from the reservoirs. In regard to channel configuration and meander pattern in Reach No. 3, it is difficult to predict changes that may occur. The maps are inadequate to clearly ascertain historical shifting of the channel. Air photos over a time span are essential to evaluate the lateral stability of the river. Also, the magnitude, frequency and duration of releases from the reservoirs must be known in order to attempt to predict future channel configuration. In qualitative terms, it appears that the channel pattern is controlled by boulders along the river banks and reduced flows will produce relatively little change. However, if aggradation takes place at the confluence, then the build-up of channel bars will deflect currents into the banks resulting in some change to the channel pattern. The pattern below the confluence is likely to have several distinct channels flowing adjacent to bars and islands.

In order to carry out further detailed analysis, however, more data are required.

ADDITIONAL DATA REQUIREMENTS

The following data are required in order to analyze the various river reaches and to predict river changes.

Data	Reach No. 2	Reach No. 3	Little Snake River
Cross sections	X	X	X
Bed material samples	X	X	X
Bank material samples	X	X	-
Water levels at various flows	X	X	X
Stage-discharge measurements	X	X	X
Reservoir Releases	X	X	-
Bed forms during high flows	X	X	-

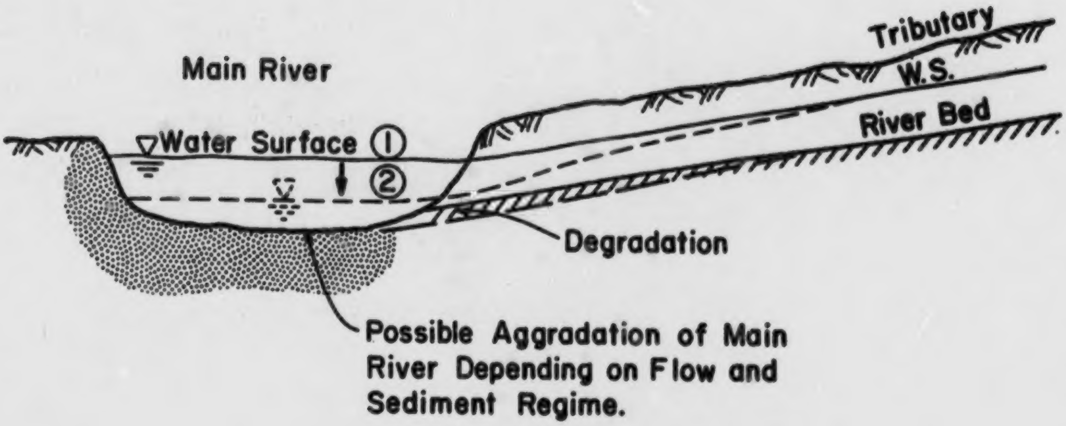
Main River
Water Surface ①
②
Tributary
W.S.
River Bed
Degradation
Possible Aggradation of Main River Depending on Flow and Sediment Regime.

Figure 3. Upstream progressing degradation on tributary caused by base level lowering on main river.

ANALYSIS OF RIVER PROCESSES AND PREDICTION OF CHANGES

With the availability of more data it may be possible to make use of available mathematical models to predict the changes to the river.

The use of the HEC-6 scour and deposition computer model developed by the U.S. Army Corps of Engineers should enable one to compute the response of the river to changes in sediment loads due to reservoir construction. It would be necessary to operate the model for a number of releases from the reservoir in order to evaluate the possible aggradation through Reach No. 3 and its possible backwater effects on Reach No. 2.

Changes to the pool and riffle sequence as well as to meandering cannot be evaluated with a mathematical model, it may be necessary to obtain data from other similar rivers that have undergone major changes in order to predict what will happen to the Yampa River.

STUDIES ON THE MORPHOLOGICAL CHARACTERISTICS OF THE YAMPA RIVER

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INTRODUCTION

The data concerning the Yampa River were made available to the author by the organizers of the workshop for carrying out a detailed morphological analysis of this river. A close scrutiny of these data and the questions posed led the author to study the following aspects of the problem.

1. Morphological characteristics of the Yampa River from Maybell to its confluence with the Green River in the absence of flood control reservoirs, with particular reference to planform and longitudinal slope.

2. Response of the Yampa River to the construction of flood control reservoirs with reference to bed level variations.

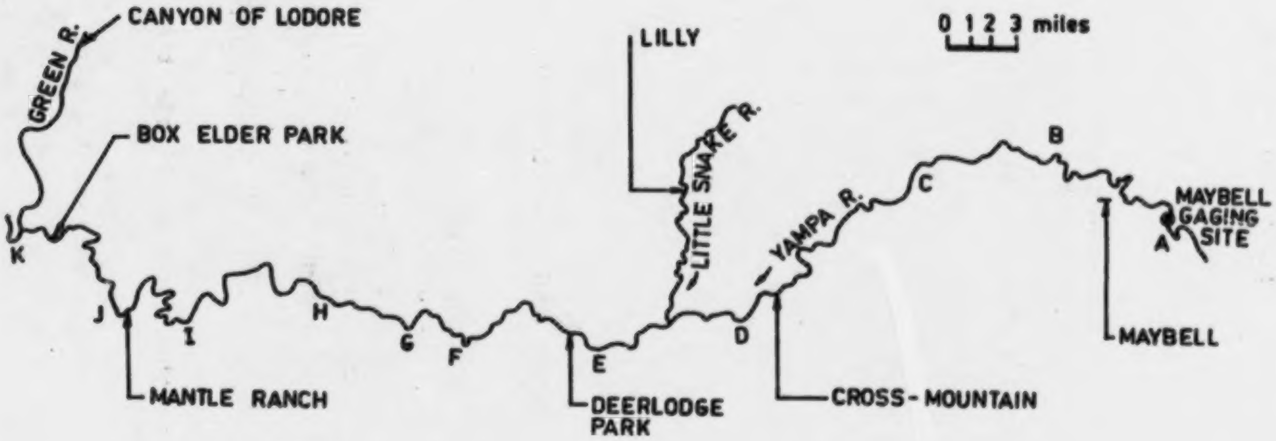
The results of the analysis are reported in the present paper; details of the data, and other information made available to the participants are omitted to avoid repetition. It would not be out of place to mention here that owing to the paucity of time and the difficulties in getting clarifications or informaton on telephone over such a long distance, the author was constrained to make several assumptions, either intuitively or on the basis of certain deductions.

PRESENT CHARACTERISTICS OF THE YAMPA RIVER

In dealing with a river, a thorough understanding of its morphological characteristics is the first essential step in the analysis of its response to manmade changes. Accordingly, several interesting features of the Yampa River are studied first.

Meander Pattern of the Yampa River Between the Little Snake and the Green River

The planview of the Yampa River along with the longitudinal bed slope for the various reaches is shown in Figure 1. The slopes have been calculated from the knowledge of elevations and distances obtained from the contour maps. It can be seen that the slope varies from 60.90×10^{-4} to 4.84×10^{-4} and that the reach under consideration is a meandering one. It was thus thought desirable to study these meander patterns in terms of meander length M_L, meander belt M_B, and the radius of curvature R. Thus, for the meander pattern shown in Figure 1, the values of M_L, M_B, and R were actually measured. In the measurement of R, it was assumed that the meander loop is a segment of a circle of equivalent radius and the best possible fit of this segment for the curved surface of the meander was used to calculate R.



REACH	AB	BC	CD	EF	FG	GH	HI	IJ	JK
SLOPE x 10^4	6.77	4.73	31.65	17.50	60.90	28.60	29.70	15.90	4.84

Figure 1. Planview of the Yampa River.

It is obvious from the geology of the area that these meanders are entrenched meanders different from the conventional meanders in alluvial plains. This contention was also supported by the finding that the observed meander characteristics did not show any correlation with the dominant discharge at Deerlodge Park. The hydrograph at Deerlodge Park was prepared by adding the mean monthly discharges of the corresponding months from the Little Snake River and the Yampa River, and the dominant discharge was taken as 8900 cfs.

The meander data were then compared with some of the available relationships for entrenched meanders. According to Bates, M_B is given by Equation 1 while Inglis suggested the use of Equation 2 for M_B.

$$M_B = 30.8\ W_s \tag{1}$$

$$M_B = 27.30\ W_s \tag{2}$$

The relationships for M_L given by Inglis and Dury are given by Equations 3 and 4 respectively.

$$M_L = 11.45\ W_s \tag{3}$$

$$M_L = 7 \text{ to } 10\ W_s \tag{4}$$

In the above equations W_s is the bankful width of the river.

In estimating the values of M_B and M_L from the above relationships, the value of W_s was taken as the width of the river. The values of M_B obtained from Equations 1 and 2 were in general 3 to 4 times larger than the actual values; on the other hand, the values of M_L obtained from Equations 3 and 4 were comparable with the actual values only for a few meander loops. Figure 2 shows the variation of M_L with R for the observed values. On this figure, the relationship between M_L and R for entrenched meanders proposed by Young (1974) has also been plotted for comparison. It can be seen that, on the average the Yampa River data follow Young's relationship.

The study of the topographic features of the Yampa River also indicates the presence of cliffs more than a thousand feet high especially on the inside bends of the entrenched meanders. Thus, the meanders are more or less confined within the high vertical walls of the river with little possibility of their lateral migration. The general meander pattern is, therefore, unlikely to be affected by any variations in discharge.

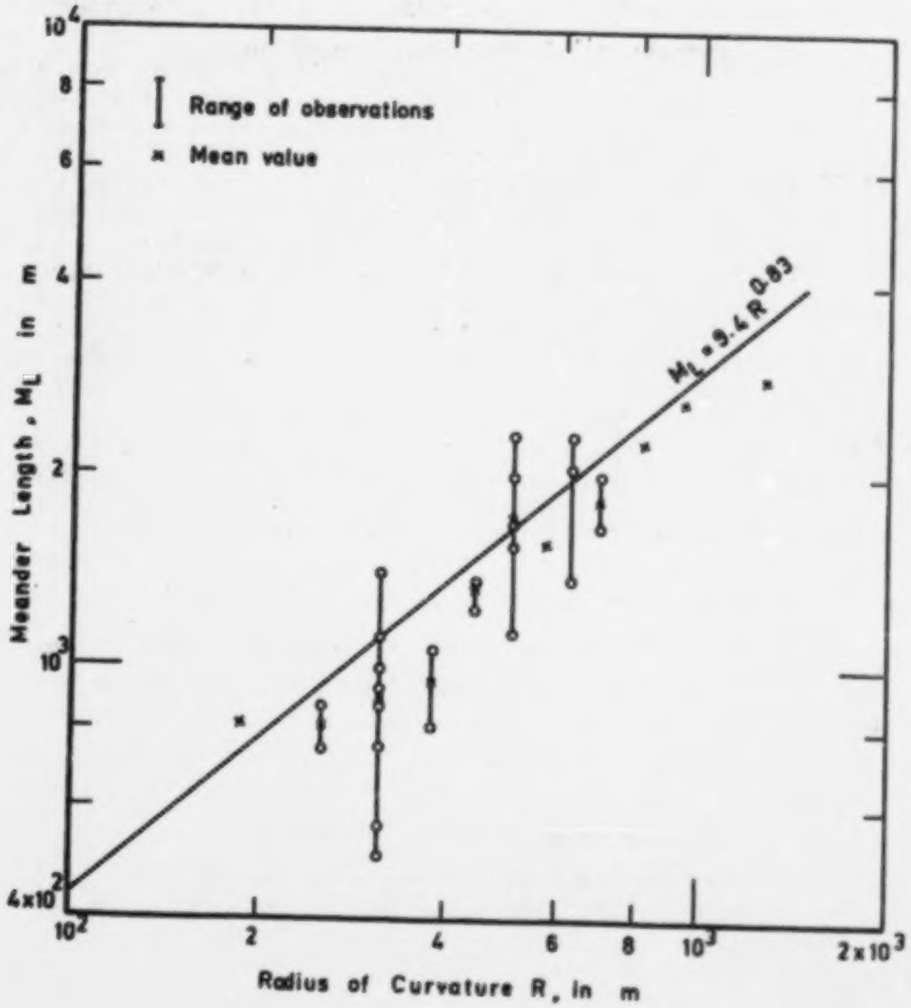


Figure 2. Variation of M_L with R for the meanders in the Yampa River.

Hydraulic and Sediment Characteristics at Maybell

The data regarding the mean monthly discharges and the maximum annual discharges of the Yampa River at Maybell have been collected since 1910. However, the gage heights are known only for the maximum annual discharges and even in this situation it is not known how these gage heights are related with the depths of flow. The sediment data include the size distribution of bed and suspended material along with the mean monthly concentration of the suspended load from 1951 to 1957 and 1977 to 1978. These data have been used to study the hydraulic and sediment characteristics of the Yampa River at Maybell.

Stage-Discharge Relationship. Figure 3 shows the stage-discharge relationship at Maybell. A close examination of this figure indicates that the gage heights corresponding to the same discharge are different for different years. Such a variation in the gage height would imply that the stream is not in true equilibrium and is either aggrading or degrading. To examine this aspect more thoroughly, the gage heights were plotted against time for different ranges of discharge as shown in Figure 4. It is interesting to note that gage heights increased continuously up to 1950 and have stabilized at constant values since then. This may be taken to be an indication of aggradation until 1950 and subsequent attainment of equilibrium conditions at its present slope.

Relation between Stage and Depth of Flow. In the absence of any definite relationship between the stage and the depth of flow, it was necessary to estimate the depth of flow for the given discharge and the slope. The size distribution of the bed material of the Yampa River is shown in Figure 5 which shows that the size of the bed material ranges from 4.0 mm to 128.0 mm with a median size of 40.0 mm. Considering the coarseness of the bed material, the bed was assumed to be flat and the value of Manning's roughness coefficient n, was computed using Equations 5 and 6 proposed by Bray (1979) and Strickler (see Garde and Ranga Raju, 1977) respectively

$$n = .104\ S^{0.177} \tag{5}$$

$$n = \frac{(d_{65})^{1/6}}{21.0} \tag{6}$$

Here S is the bed slope and d_{65} is the size of the bed material in meters such that sixty-five percent of the material is finer than this size.

The values of n obtained from Equations 5 and 6 were 0.0285 and 0.029 respectively and thus a constant value of n = 0.03 was adopted in the subsequent calculations.

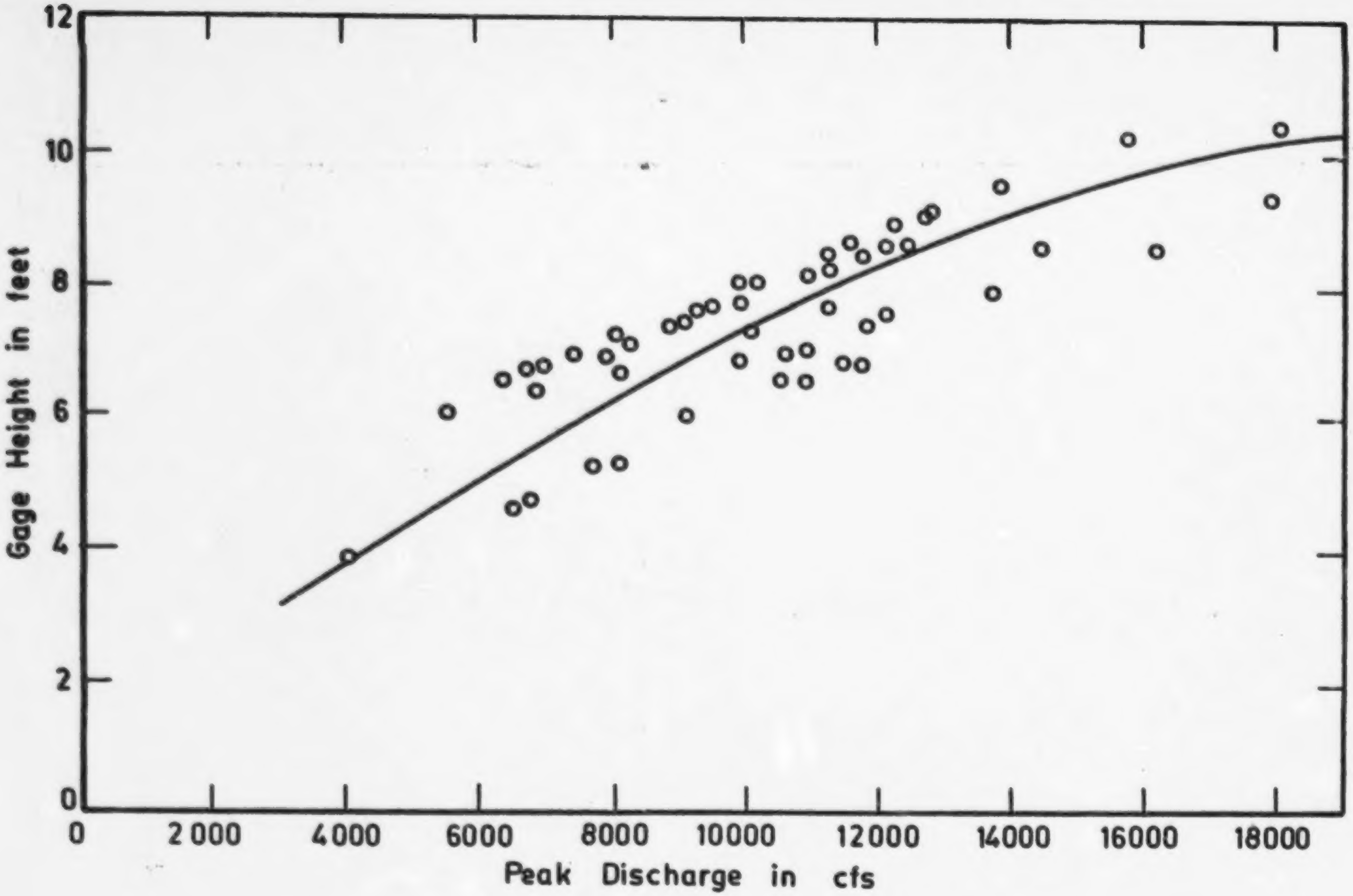


Figure 3. Relationship between stage and discharge for annual-maximum flows at Maybell.

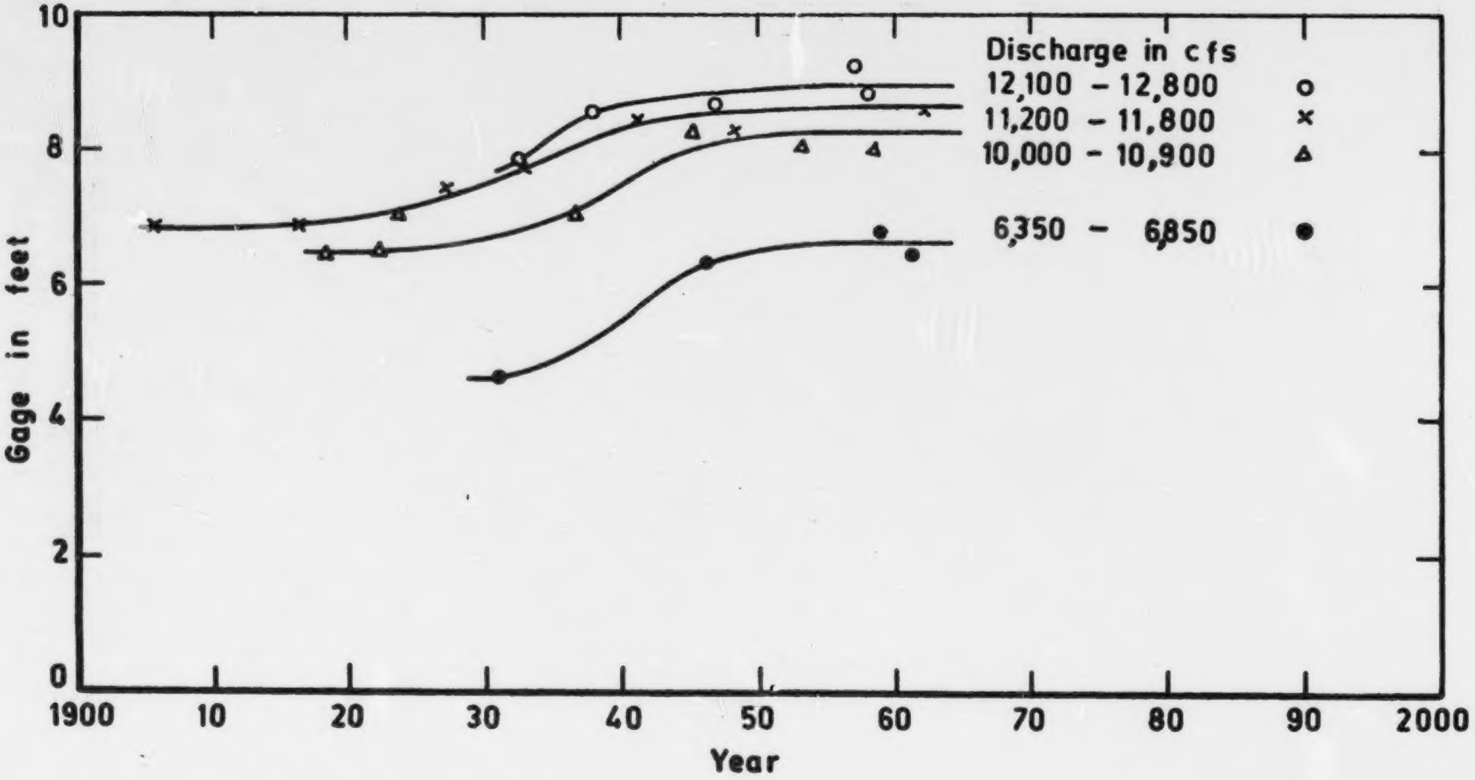


Figure 4. Variation in river stage with time for different discharges.

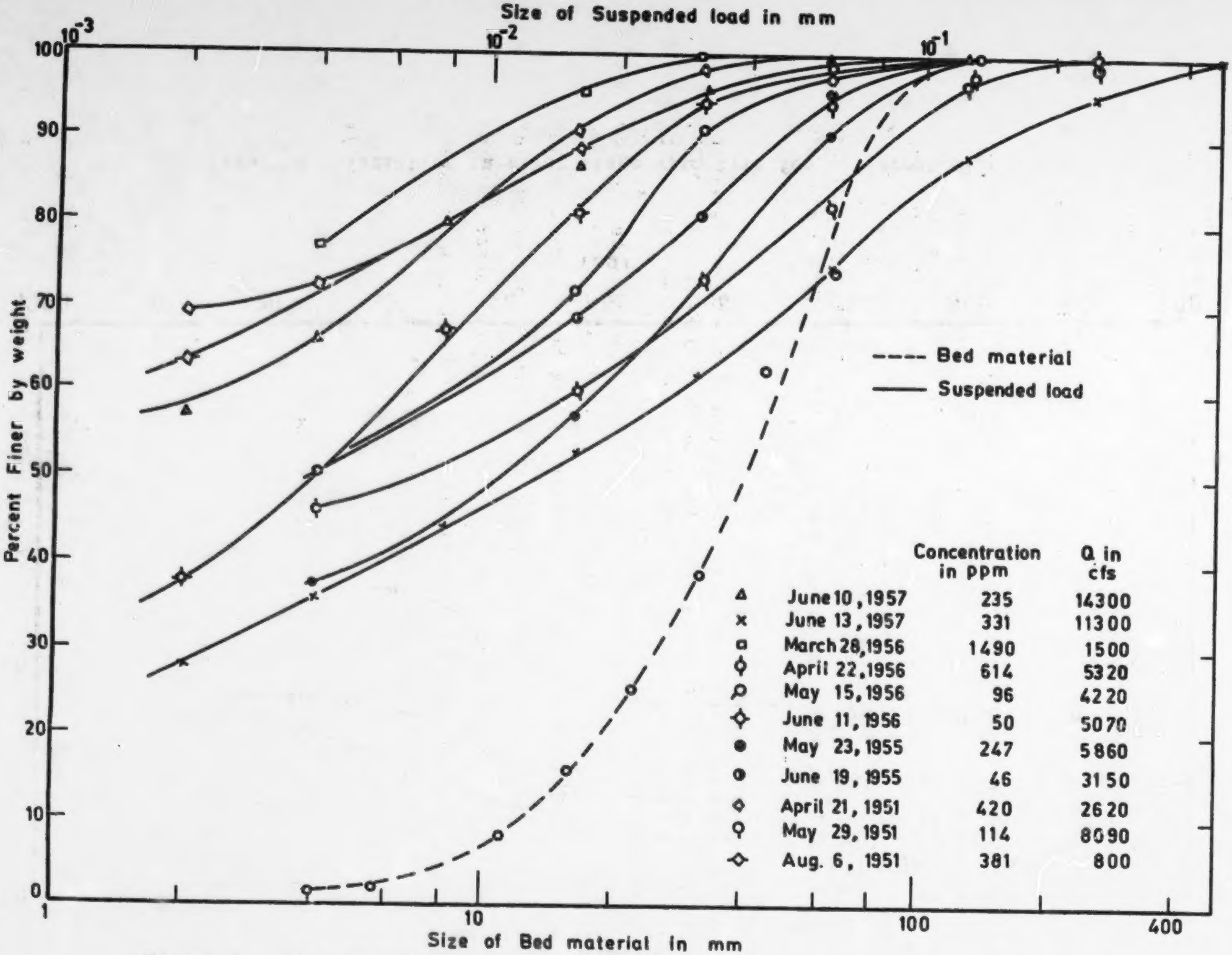


Figure 5. Size distribution of suspended load and bed material in the Yampa.

The Manning's equation was used to calculate the depths of flow for the peak flows. The river cross section was taken to be a wide rectangle with an average bed width equal to 326 feet. The depths so computed were compared with the corresponding gage heights. This comparison showed that around the 1920's the gage heights and the depths of flow were more or less the same; however, after 1950 the indicated gage heights were consistently 1.40 feet higher than the estimated depths.

The foregoing finding substantiates the conclusion that aggradation of the stream stopped after 1950.

Sediment Load Computations. Figure 5 shows the size distribution of the suspended load for the years for which the data were available. On this plot the size distribution of the bed material is also shown. It is evident from Figure 5 that the suspended material is much finer than the bed material. Further, with increase in shear stress or discharge the concentration of the suspended load and its median size normally increase when the suspended load happens to be a part of bed material load. However, such is not the situation in Figure 5 and this is taken to indicate that the observed suspended load is, in fact, wash load.

For the estimation of bed load, one can use either Meyer-Peter and Muller's equation or follow Einstein's (see Garde and Ranga Raju 1977) procedure of computation. Recently some work has been carried out by Misri (1980) concerning the partial transport of bed load in case of highly nonuniform material and this has been used to estimate the bed load. According to Misri the parameter i_B/i_b is related to τ_o/τ_{ci} as shown in Figure 6. Here i_b is the fraction of the bed sediment in the given size range d_i, i_B is the fraction of the bed load transport of size d_i, τ_o is the grain shear, and τ_{ci} is the critical shear corresponding to d_i. In the present study Figure 6 in conjunction with Meyer-Peter's Equation has been adopted to estimate the bed load.

$$\phi_i = 8(\tau_{*i} - .047)^{3/2} \qquad (7)$$

In Equation 7, parameters ϕ_i and τ_{*i} are defined as,

$$\phi_i = \frac{q_{B_i}}{\gamma s}\frac{1}{\sqrt{(\frac{\rho_s}{\rho_f} - 1)gd_i^3}} \quad \text{and} \quad \tau_{*i} = \frac{\tau_o'}{(\Delta\gamma_s)\, d_i}$$

Here q_{B_i} is the bed load transport rate in lb/ft of size d_i, γ_s is the unit weight of sediment, $\Delta\gamma_s = (\gamma_s - \gamma_f)$, γ_f is the unit weight of water, ρ_s and ρ_f are the mass densities of sediment and water respectively, and g is the acceleration due to gravity.

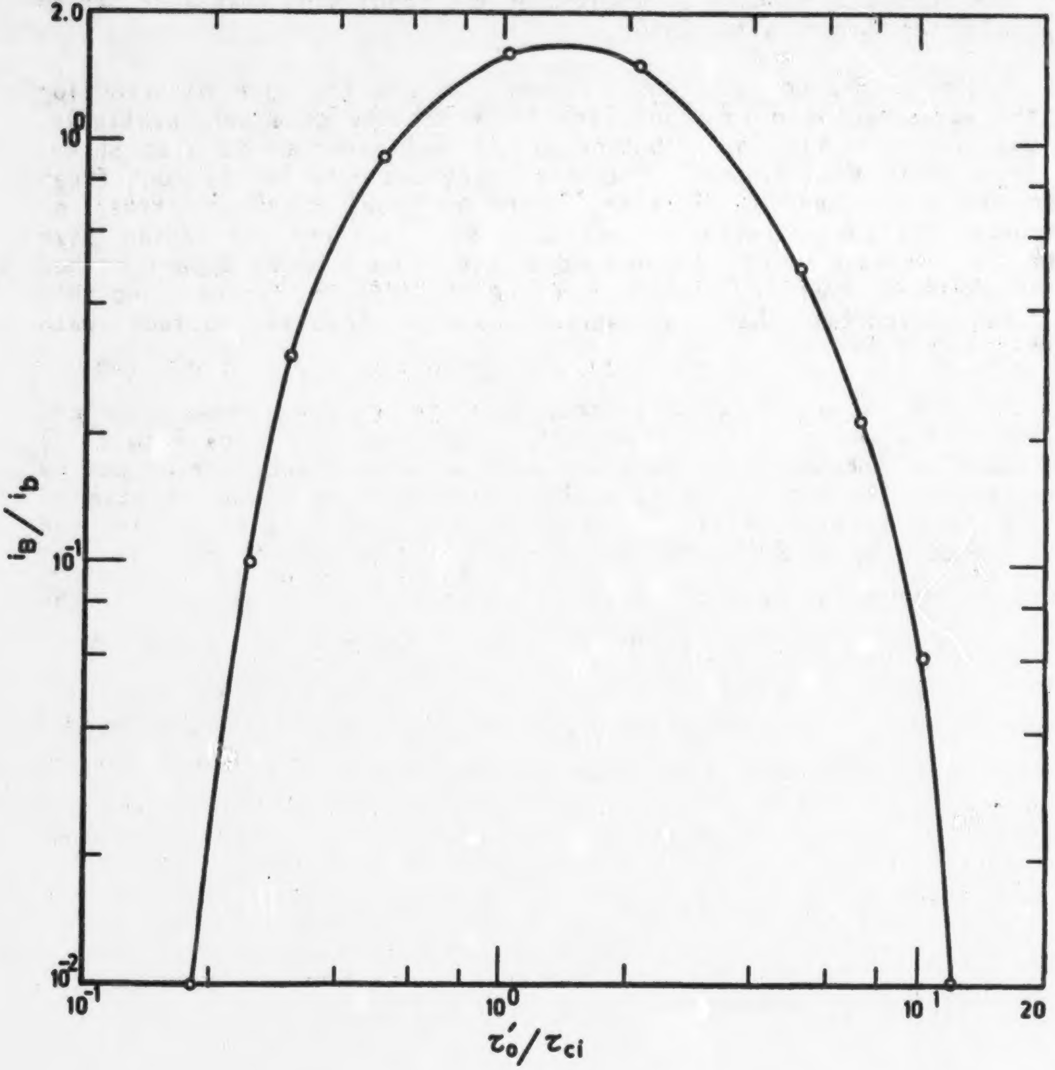


Figure 6. Variation of i_B/i_b with τ_o'/τ_{ci}.

The values of τ_{ci} were determined from Shields' (see Garde and Ranga Raju, 1977) and τ_o was taken as the total shear τ_o in view of the earlier assumption of flat bed.

Using Figure 6 in conjunction with Equation 7, the bed load transport for each size range d_i was calculated as $i_B \, q_{B_i}$ in lb/ft s. The total bed load Q_B in T/day was then determined in the following manner.

$$Q_B = \Sigma \, i_B \, q_{B_i} \text{ x width of the channel in ft x } 38.57.$$

The bed load transport rates thus obtained are shown in Table 1.

Table 1. Transport rates of the Yampa River at Maybell.

Average shear stress τ_o in lb/ft²	0.104	0.137	0.163	0.208	0.239	0.307	0.400
Bed load Q_B in T/day	6.35	54.30	135.7	341.6	625.0	1358	2000

The above hydraulic and sediment computations have been used to study the longitudinal bed profile of the Yampa River at Deerlodge Park. Following is a brief description of this study.

Longitudinal Bed Profile of the Yampa River Near Deerlodge Park

The longitudinal bed slope of the Yampa River near Deerlodge Park is 1.75×10^{-3} as indicated in Figure 1. This slope is much steeper than the slope of 6.77×10^{-4} at Maybell. In general the stream slope decreases in the downstream direction. The converse situation prevailing here led the author to examine the possible reasons. Since the Little Snake River brings in about 70% of the total sediment load (mostly fine suspended load) of the Yampa River downstream of the confluence, occurrence of aggradation downstream of the Little Snake is a possibility. In making the necessary computations the following assumptions have been made.

1. Bed load of the Little Snake River has been taken from the available relationship at Dixon, while discharges and the suspended load data have been taken from the records at Lily.
2. The river cross section and the bed material characteristics at Deerlodge Park have been assumed to be the same as at Maybell.

3. Variations in discharges and the sediment load in the Yampa River from Maybell to its confluence with the Little Snake River have not been considered in the absence of any relevant information in this regard.

4. The median size of the suspended load is taken as 0.04 mm.

The total discharge, bed load, and suspended load at Deerlodge Park were then estimated by adding the corresponding values for the Yampa and the Little Snake Rivers.

Aggradation in the Yampa River near Deerlodge Park will take place if the incoming sediment load exceeds the equilibrium transport rate. As mentioned earlier, the suspended load from Maybell as well as from the Little Snake River is, in fact, wash load. Therefore, one could use Pullaiah's (1978) results for determining the slope required to transport the incoming wash load without objectionable deposition. According to Pullaiah, the limiting concentration C in ppm is related to $u_{*b} S/w_o$ as shown in Figure 7. Here u_{*b} is the shear velocity corresponding to bed, S is the bed slope, and w_o is the fall velocity for the median size of the wash load. In the present case, since the river is wide, u_{*b} is taken as u_*. Using Manning's equation u_*S/w_o may be expressed as

$$\frac{1}{w_o}\left(\frac{Q\ n\ g^{5/3}}{1.49\ B}\right)^{0.30} S^{1.35}$$

Available data for the year 1977-1978 indicate that the Little Snake River had a maximum concentration of wash load on September 22, 1978. The concentration was 17,300 ppm with a discharge of 123 cfs. For the corresponding period, the values of wash load concentration and discharge for the Yampa River were 91 ppm and 226 cfs respectively. Thus, a total discharge of 349 cfs with a wash load concentration of 6161.52 ppm was considered at Deerlodge Park.

For $Q = 349$ cfs, $S = 6.77 \times 10^{-4}$, and median sediment size of 0.04 mm for the wash load, Figure 7 gives the limiting concentration as 3500 ppm. Since the combined concentration from the Little Snake and the Yampa River is 6161.52 ppm, the river would need a steeper slope to carry this load. Figure 7 gives the value of this slope as 1.315×10^{-3} for $C = 6161.52$ ppm. Thus, the Yampa River near Deerlodge Park would aggrade until this slope is attained.

The slope estimated above is smaller than the present slope of the reach. While the approximate nature of Figure 7 could be partly responsible for this difference, the possibility that the Little Snake River carried higher sediment loads in the past (leading to steeper slopes in

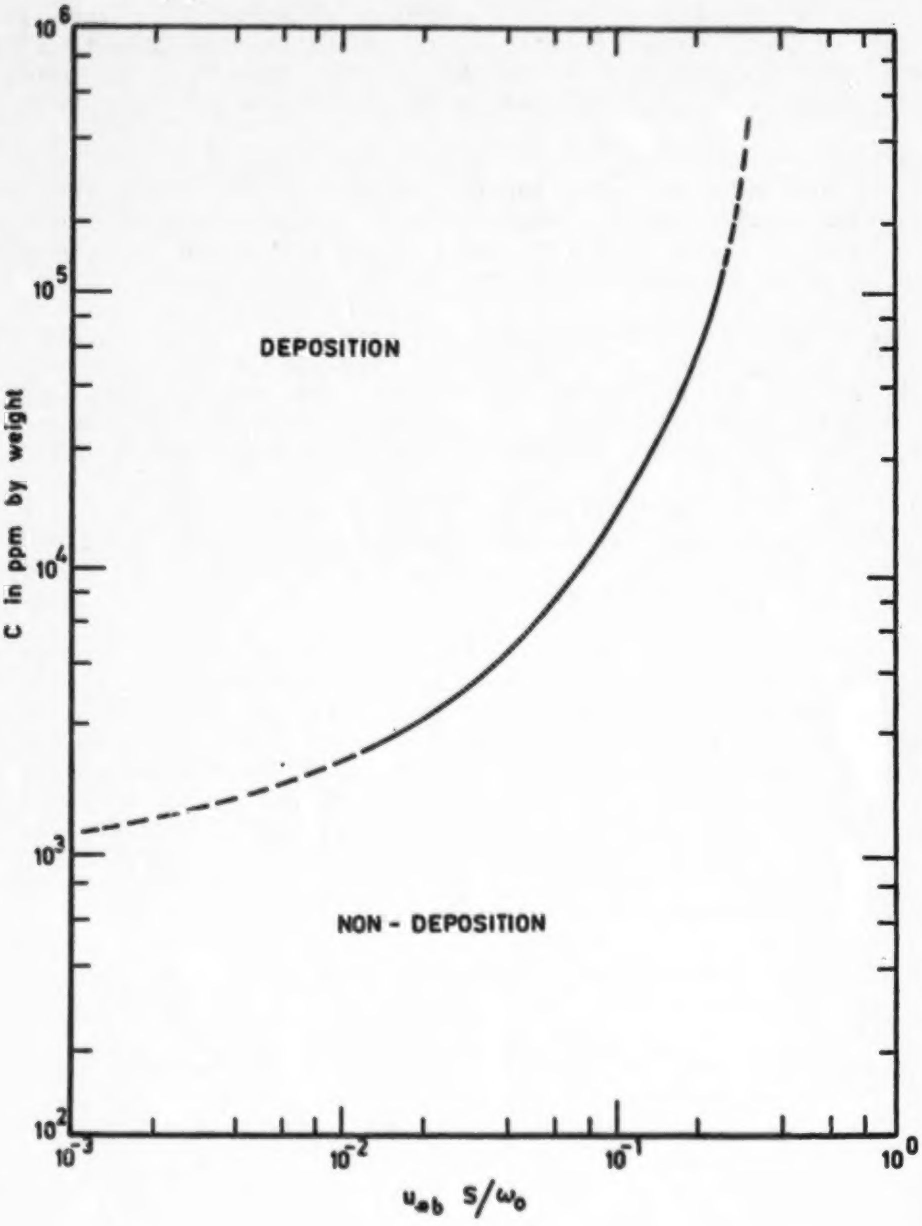


Figure 7. Variation of C with $u_{*b}S/w_o$.

the Yampa River) cannot be ruled out. With aggradation in the Yampa River, some of the wash load would start depositing in the Little Snake River itself, thus causing a reduction in wash load entering the confluence.

Conceding that a slope of 1.75×10^{-3} downstream of the confluence is a result of deposition of fine sediment brought in by the Little Snake River, it would now be interesting to examine the effects of the varying discharges and sediment loads on the stability of this reach. The data for the year 1977-1978 were used in this analysis and these are listed in Table 2.

It is evident from this table that during 1977-1978, the total sediment load supplied at Deerlodge Park was less than the corresponding carrying capacity. One would thus expect degradation to occur provided the composition of the bed does not change.

Table 2 also shows that the bed material is likely to move from March 1978 to July 1978 because during this period shear exceeds the critical value based on Shields' criterion. The maximum size that is likely to move during this period is 38.0 mm. Thus, due to the removal of finer sizes from the bed, the bed would eventually become armored.

From the above qualitative analysis, it appears that the present top surface of the Yampa River near Deerlodge Park is a paved one and the sediment load supplied from the upstream is being carried through without any bed level variations.

The analysis so far presented deals with the present characteristics of the Yampa River. However, these characteristics are likely to be modified if flood control reservoirs are constructed on the upstream reach. In fact construction of a large number of small flood-retaining reservoirs on the Yampa River is envisioned. Obviously, one would then like to know the response of the Yampa River to the construction of these reservoirs with particular reference to bed level variations. Therefore, the subsequent discussion deals with the change in bed characteristics at Deerlodge Park as a result of construction of Cross Mountain reservoir the location of which is shown in Figure 1.

RESPONSE OF YAMPA RIVER DOWNSTREAM OF CROSS MOUNTAIN RESERVOIR TO THE CONSTRUCTION OF THE DAM

The available data indicate a dam of about 360 ft. height with a capacity of 10.64×10^{6} acre ft located at Cross Mountain on the Yampa River immediately upstream of its confluence with the Little Snake River (Figure 1). This reservoir is likely to trap all the bed and wash load that is supplied to it by the Yampa River at Maybell. Therefore, the releases from this reservoir will be sediment free. These releases along with the inflow from the Little Snake River will affect the present bed levels at Deerlodge Park.

Table 2. Hydraulic and sediment characteristics at Deerlodge Park during 1977-1978.[a]

S. No.	Period (1)		Total mean monthly Q cfs (2)	Bed load Q_B T/day		Wash load Q_S T/day		Max. size of bed material likely to move in mm (7)
				Incoming (3)	Carrying Capacity (4)	Incoming (5)	Max. carrying capacity without deposition (6)	
1	Oct	1977	199.44	-	-	94.89	5385.0	-
2	Nov	1977	243.50	-	-	69.56	6903.0	-
3	Dec	1977	306.12	-	-	256.48	9340.0	-
4	Jan	1978	323.87	-	-	82.20	10056.0	-
5	Feb	1978	401.22	-	-	85.86	14083.0	-
6	Mar	1978	1125.02	-	80.0	8666.00	57714.0	9.30
7	Apr	1978	4070.60	80.0	1500.0	7615.66	417643.0	20.10
8	May	1978	9672.0	2320.0	5000.0	22529.54	1984700.0	33.80
9	June	1978	11917.66	2730.0	6000.0	9684.00	2960300.0	38.30
10	July	1978	3442.81	58.0	1000.0	1048.38	316000.0	18.20
11	Aug	1978	612.55	-	-	693.12	25100.0	-
12	Sept	1978	265.90	-	-	1216.26	7753.0	-

[a] (2) Addition of mean monthly flows of the Yampa and Little Snake Rivers.
(3) Addition of bed loads rates from the two rivers at Dixon and Maybell respectively.
(4) Computed using Figure 6 and Equation 7.
(5) Addition of observed suspended loads at Lily and Maybell.
(6) Computed using Equation 7.

The data for the year 1977-1978 were once again used to study the bed characteristics at Deerlodge Park as discussed above. Table 3 shows the hydraulic and sediment characteristics at Deerlodge Park considering the sediment free releases from Cross Mountain reservoir.

Table 3. Bed level variations at Deerlodge Park after construction of Cross-Mountain Reservoir.[a]

S. No.	Period (1)	Total mean monthly discharge Q in cfs (2)	Bed load Q_B T/day Incoming (3)	Bed load Q_B T/day Carrying capacity (4)	Wash load Q_S T/day Incoming (5)	Wash load Q_S T/day Max. carrying capacity without deposition (6)
1	Oct. 1977	471.70	-	-	40.0	17830.26
2	Nov. 1977	402.60	-	-	59.0	14131.26
3	Dec. 1977	398.80	-	-	250.0	13998.0
4	Jan. 1978	334.00	-	-	77.0	10821.60
5	Feb. 1978	324.30	-	-	80.0	10507.32
6	Mar. 1978	801.25	-	24.0	8550.0	37859.0
7	Apr. 1978	1100.30	-	87.0	3760.0	56445.0
8	May 1978	4790.0	1900	2000.0	15426.0	620784.0
9	June 1978	5705.0	1940	2700.0	5555.0	739368.0
10	July 1978	3122.2	-	864.0	393.0	252898.0
11	Aug. 1978	1182.2	-	102.0	505.0	60646.0
12	Sept. 1978	698.0	-	7.00	936.0	30153.6

[a] (2) Addition of mean monthly flows of the Little Snake River and the corresponding releases from the Cross Mountain reservoir.

(3) From the Little Snake River only as releases from the Cross Mountain reservoir are assumed to be sediment free.

(4) Computed using Figure 6 and Equation 7.

(5) From the Little Snake River only as releases from the Cross Mountain reservoir are assumed to be sediment free.

(6) Computed using Figure 7.

As was the circumstance before the construction of the reservoir, the wash load would be safely transported by the stream without any deposition (Table 3). The bed material load is, however, slightly smaller than the carrying capacity. This indicates the possibility of degradation downstream of the Cross Mountain reservoir. For reasons mentioned earlier, this degradation will also not result in any significant bed level variation; the surface will be paved after enough fine material has been picked up by the flow, but after the surface is paved the material that comes in from the Little Snake River would be safely carried down by the Yampa River at Deerlodge Park.

The Cross Mountain Dam would also cause aggradation on its upstream side. The amount and the extent of aggradation would depend on the capacity of the reservoir, the length of the back water profile, the amount and the nature of the total sediment load supplied to it from Maybell. The process of aggradation in the reservoir would be accelerated as a result of surface mining which is contemplated upstream of Maybell. The data indicate that the net effect of surface mining will be to increase the total sediment load of Maybell by 7%. With available methods, it would have been interesting to study the deposition profiles upstream of dam; however, due to paucity of time, the process of aggradation could not be studied in this paper.

CONCLUSIONS

The available data on the Yampa River have been analyzed to study the morphological characteristics of the river. The analysis reported in this paper is based on some simplified assumptions made either intuitively or on the basis of certain deductions. The main findings are as follows.

1. The meanders in the Yampa River are entrenched and are unlikely to be changed by the variations in discharge and sediment load brought about by the construction of dams.

2. The Yampa River at Maybell was an aggrading river until 1950 and appears to have attained equilibrium conditions after that.

3. The slope of the Yampa River near Deerlodge Park is steeper than in the upper reaches on account of aggradation caused by the high influx of sediment from the Little Snake River.

4. The Cross Mountain reservoir is unlikely to affect the bed levels in the Yampa River downstream of the dam.

ACKNOWLEDGEMENT

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YAMPA RIVER

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INTRODUCTION

The objective of this study is to comment on the downstream effects on channel morphology of the Yampa River due to two proposed flood retaining reservoirs. The prediction is to be made on the basis of a package of background material provided by the U.S. Fish and Wildlife Service. Two short reaches of the Yampa River, for which some survey data are available, are to be considered specifically.

A quick review of the data package indicated that only a relatively small part of the information that one would need for any kind of quantitative prediction is presently available. Before the workshop, it was therefore only possible to state some general, qualitative conclusions and to list the types of data needed for more refined estimates. The present text was revised after the workshop to take data obtained there (e.g., air photos) into account and to refer to some important references seen since completion of the preconference text.

DEFINITION OF THE PRESENT RIVER REGIME

Discharge

The Yampa River appears to have a strongly seasonal discharge regime, dominated by snowmelt runoff during spring and early summer. Low flows prevail during the rest of the year, with the annual low occurring either in late summer or in midwinter.

For proper impact assessment the following discharge data should either be available or be estimated for each reach of interest.

1. Flood frequency.
2. Low flow frequency.
3. Typical hydrographs.
4. Flow duration curve.
5. Flow probability curves, giving the estimated distribution of flow for every day of the year.
6. Tabulation of mean monthly flows.

In the present case only items 1, 2, and 3 are available and only for one of the two reaches (Lily Park).

Sediment Transport

Long term suspended data are available for the upper reach and for the Little Snake River, a major tributary between the two test reaches. The computed bed load estimates given by Andrews (1978) are, in the writer's view, probably considerably too large. The size distribution of the suspended load is reasonably well defined. It appears to consist predominantly of clay and silt. Concentrations of 1000 mg/l, or higher, are being observed occasionally, indicating a potential for density currents in the proposed reservoirs. The bed load size distribution is not given.

As in the case of discharge, it would be desirable to have better site specific data. Bed load computations in particular cannot be transposed from reach to reach. They should be carried out for all the reaches of interest.

Bed and Bank Materials

A single bed material sample is provided for the Yampa River, but it was not taken within either study reach and no information is provided about sampling method or the details of the sample site. Indirect evidence, such as a map profile, air photos, and reports by others indicate that the Lily Park Reach has basically a gravel bed, but with extensive sand deposits within the channel zone. The bed materials of the Box Elder Reach are probably also mainly gravel, but there may also be some cobbles and boulders.

Extensive site-specific bed and bank material sampling is required for proper definition of the existing conditions. For each sample both the sampling method and the sedimentary environment of the sample site should be documented carefully. Kellerhals and Bray (1971) discuss sampling procedures and a check list developed by Bray (1972) for identification of the sedimentary environment of bed material samples is appended to this report (Appendix A). The list was developed for a general morphologic study of rivers in Alberta and may need to be modified depending on local conditions and on study objectives.

If there is a possibility of degradation, as in the present case, sub-bed materials are of interest. Any drill logs from within or near the study reach should be examined.

Hydraulic Geometry

The cross sections provided for the two study reaches are of little use as they cover only very short reaches and do not include water levels. The location of the sections is also not shown on any plan. Based on a rough map profile (Figure 1) the two reaches appear to have slopes of approximately 0.001.

Five to 10 cross sections along a reach of at least one, but better, two to three meander cycles, are needed for a reasonable definition of hydraulic geometry. At least five water level observations covering a wide range of discharge should be available at each section

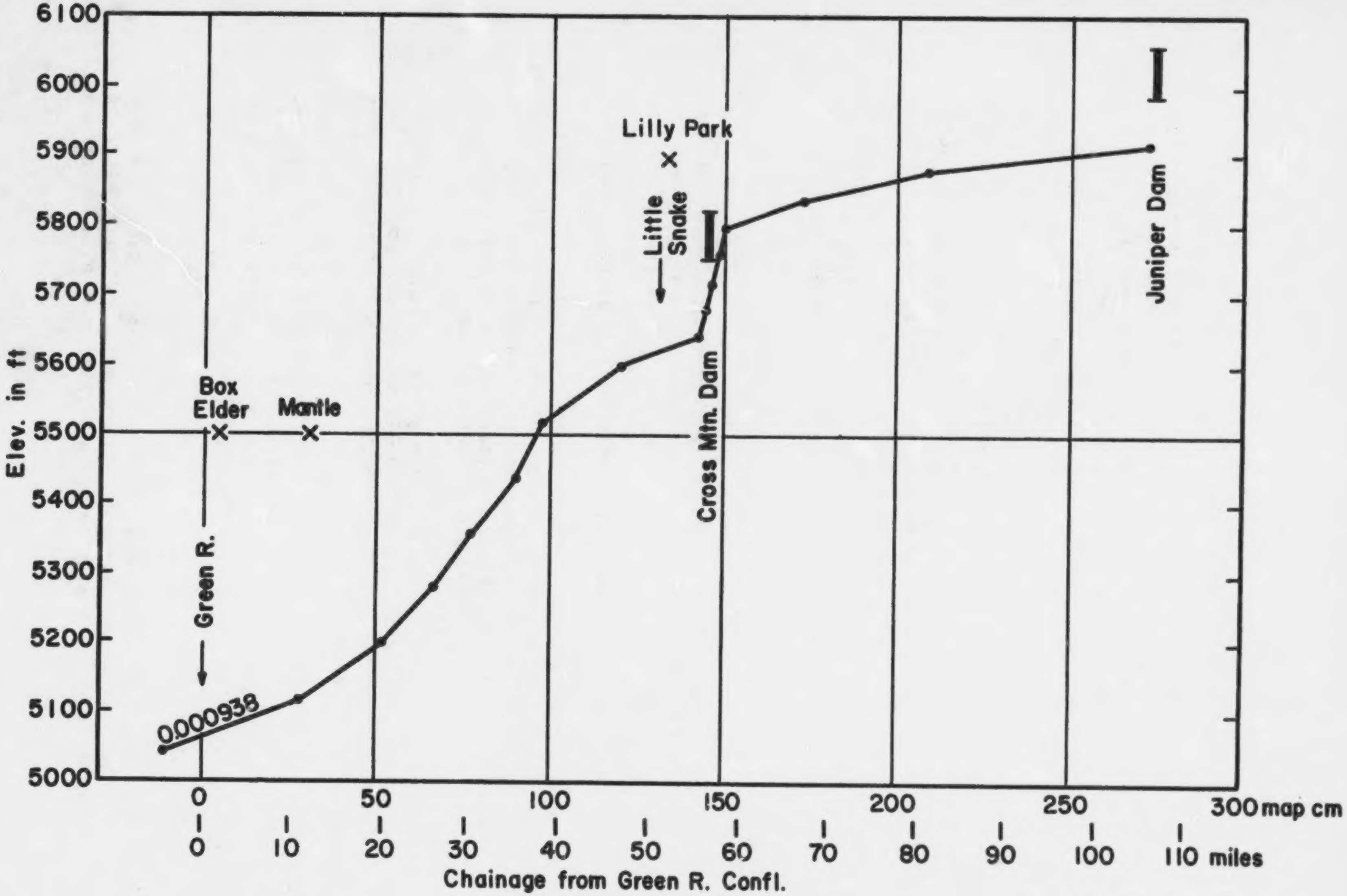

Figure 1. Yampa River profile.

so that the information on the time distribution of flows can be converted to stage.

Channel Morphology and Fluvial Processes

With only topographic mapping at 1:62500 little could be said about the detailed channel morphology. The channel profile at Figure 1 indicates strong bedrock control along much of the lower Yampa River, although the two surveyed reaches are probably alluvial. Air photos seen during the course of the workshop confirm that the Lily Park Reach is essentially alluvial. It is partially confined, with an irregular meander pattern and occasional islands.

For an adequate morphological assessment air photo analysis and field inspection are essential. Kellerhals, et al. (1976) discuss in detail the types of information that one looks for and provide check lists to help assure that nothing of importance is missed.

Fluvial process rates (e.g. channel shifting, bar migration rates), and any nonequilibrium trends (e.g., channel zone widening, entrenchment,) can often be obtained from comparative analysis of old and recent air photos or maps.

Other Matters

A wide range of water quality parameters is obviously of interest but falls outside the objectives of the present workshop. One needs to be aware, however, that there are linkages between water quality and channel morphology. Water temperature, for instance, can be greatly affected by changes in channel zone width or by changed bank vegetation.

Bank vegetation also affects channel stability, fisheries, recreational values, flood stages and others. The existing bank vegetation should be carefully documented and checked for long term trends. Comparative analysis of old and recent air or ground photos and possibly local interviews are needed to detect long term trends.

In northern climates the ice regime may have a major influence on channel morphology (Kellerhals and Gill 1973, Kellerhals and Church, in press). Since dams can drastically alter the existing ice regime, this becomes a matter of great importance.

The water exchange between a stream channel and the surrounding valley aquifer can be important for fisheries and for water users. Since it, also, is potentially affected by dams, the existing situation needs to be documented as best as possible.

PROJECT DESCRIPTION

The two proposed dams are only described in a very general manner and the data on future, regulated flows are not consistent with the natural flow data. Tables of average monthly reservoir releases for the period 1927-1976 are provided and indicate considerable regulation. The highest monthly flows are reduced by around 50. The effect on flood flows is unknown but a significant reduction appears probable.

For a detailed impact assessment it would be desirable to have the regulated flow regime defined in a similar manner as the natural regime. In the case of peaking plants, hourly rather than daily flow distributions may be needed. The physical limits on flow releases are also of interest. Experience has shown that, regardless of what a water license says, projects will occasionally be operated at their limits due to emergency situations (e.g., a transmission line failure might result in zero discharge unless there is a fail-safe minimum release system). The reservoir filling schedule may also need to be considered.

The thermal structure of the proposed reservoir and the elevation of the various outlet works are important because they will determine whether significant amounts of suspended load might pass through the reservoir. They naturally also determine the downstream water temperature and ice regimes.

DOWNSTREAM EFFECTS

Basis for prediction

The interrelations between an imposed flow regime with sediment supply and the resulting channel morphology are only very poorly understood at present, and what little quantitative information is available is generally based on empirical analysis of field data rather than on an understanding of the basic physical processes involved. However, river regulation has now been going on for such a long time that, when considering a new project, it is generally possible to find comparable case histories. They are needed primarily to identify potentially significant impacts. After the basic morphological impacts have been identified, quantitative estimates are sometimes possible (e.g., degradation between rigid banks, vegetation encroachment into a stable channel zone), but in many situations an empirical analysis of case histories remains the only basis for quantitative prediction (e.g., change in channel pattern, change in bar morphology).

Effects on the Yampa River

Assuming that the main structure of the Yampa River channel consists of gravel or coarser materials, combined with local bedrock outcrops, the dams are unlikely to cause major morphological changes. The material in the major bars and riffles probably moves only infrequently now and it might never move under regulated conditions.

Along the Lily Park Reach which lies upstream of the Little Snake River confluence, the river bed surface will be winnowed of fines and transformed into a very stable, coarse armor layer. There may be some minor degradation if the highest flood releases can move some of the bed gravel.

If the seepage gradient at the channel bed is predominantly downward (out of the channel), the stability of the armor layer, combined with minor amounts of fines passing through the reservoir could lead to a totally sealed channel. This could be detrimental to fish spawning.

Reduced floods generally lead to a tendency towards reduced channel size. Upstream of the Little Snake River, the Yampa River lacks the sediment supply needed for a fast reduction in channel size. On some of the higher bar surfaces and parts of the river bank trees and shrubs will grow and might then very slowly silt in. With the high suspended sediment load of the Little Snake River, areas of vegetation encroachment downstream of the confluence will quickly aggrade to flood plain level. The canyon reaches of the Yampa Rver are probably too steep for any significant suspended load deposition, but there will still be extensive vegetation encroachment, as shown in a recent, well documented study of the Colorado River below Glen Canyon Dam (Turner and Kariscak 1980). Vegetation encroachment reduces solar energy input to the water, increases water losses, and increases flood levels for any given discharge.

The most visible morphological changes are likely to occur in the vicinity of the Little Snake River confluence. Any alluvial confluence region represents a delicately balanced equilibrium between the long-term sediment transporting capacities of the various channels. In the present situation it appears reasonably certain that the reduced gravel transport capacity of the Yampa River will be the dominant effect, outweighing any effect due to reduced gravel inflow to the confluence from the Yampa River. The confluence area will gradually aggrade until the downstream Yampa River has enough slope to move the incoming load. Initially the Little Snake River might degrade somewhat due to reduced backwater from the Yampa River.

Aggradation at tributary confluences is a general result of regulation or flow reduction (Kellerhals et al. 1979) and it will eventually result in a more irregular profile for the Yampa River. In a recent paper Graf (1980) shows how regulation of the Green River is gradually making rafting more difficult due to increasing severity of the rapids. Vegetation encroachment and winnowing of fines in the remaining channel zone are also making it more difficult to find campsites in the Green River canyon.

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APPENDIX A

RESEARCH COUNCIL OF ALBERTA
HIGHWAY AND RIVER ENGINEERING DIVISION
UNIVERSITY OF ALBERTA
NO. DES 2/71
DEPARTMENT OF CIVIL ENGINEERING

RIVER DATA SHEET

DESCRIPTION OF BED-MATERIAL SAMPLING SITE, CODES

Sample No.	Type	Location to W.S.	Geomorphic Unit	Loc. on Geo. Unit	Lateral Location	Longitudinal Location	Direction	Pavement	Activity	Representa-tiveness	Particle Shape	Particle Lithology

NOTE: Use -1 for "unknown" and use multiple codes where necessary.

```
SAMPLE NO.:  Station  -  Year  -  Type  -  No.          TYPE:  Grid-by-number = N
    (e.g.):    10          70       N         3                Photo          = P
                        (10-70-N3)                             Sieve          = S

                             Location in Relation to                                       Location on
Sample Type                      Water Surface                  Geomorphic Unit           Geomorphic Unit
Lateral Location in Channel
1  tape grid                 0      at edge of water            0  no particular feature  0  not applicable        0  not
applicable
2  stepping (toe) grid       1 (n) n ft above water surface     1  side bar               1  near upstream end     1  near left
bank
3  grid photo                2 (n) n ft below water surface     2  point bar              2  near centre           2  near left
quarter point
4  sieve, including surface                                     3  mid-channel bar        3  near downstream end   3  near centre
5  sieve, excluding surface                                     4  island                                          4  near right
quarter point
                                                                5  in pool                                         5  near right
bank
                                                                6  at riffle
                                                                7  at cross-over

Longitudinal Location        Direction of Linear Grid Samples      Pavement Code                          Activity Code
1  relatively straight reach 0      not applicable                 0  no apparent pavement                0  no indication of
recent movement
2  bend to left              1      parallel to flow               1  only fines removed from surface     1  indication of recent
minor movement
3  bend to right             2      perpendicular to flow          2  obviously larger material at        2  indication of recent
major movement
                             3 (d) at d degrees to flow                surface

Representativeness Code                 Predominant Particle Shape                  Predominant Particle Lithology
0  not representative of site           1  spherical and rounded                    1  granite      5  slate       9  conglomerate

1  estimated average for site           2  elongate and rounded                     2  gneiss       6  schist      10
sandstone
2  coarsest material at site            3  angular                                  3  basalt       7  limestone   11  shale
3  finer than average                   4  platy                                    4  quartzite    8  dolomite
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DOWNSTREAM RIVER MORPHOLOGICAL CHANGES FROM PROPOSED RESERVOIR CONSTRUCTION ON THE YAMPA RIVER, COLORADO

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INTRODUCTION

River system processes are closely related to the dynamics of the ecosystem. Streamflows, sediment transport rates, and channel morphology all effect the instream habitat and each can be altered by river utilization activities. Man's activities, such as diversions or reservoir construction, can alter the channel morphology enough to affect fisheries and other aquatic biology. For a proper river utilization plan, possible morphological changes in both short and long terms should be anticipated. This paper reviews the data needs, suggests the methods of analysis, and presents a qualitative assessment of the downstream river morphological changes from proposed reservoir construction on the Yampa River, Colorado.

The Yampa River is located in northwestern Colorado. The river is essentially a gravel-cobble bed stream upstream of the confluence with the Little Snake River. Below the confluence the river is a mixture of sand and gravel bed stream. Several small flood control reservoirs are to be constructed on the Yampa River; one is the proposed Cross Mountain Reservoir and another is the proposed Juniper Reservoir. These reservoirs will impose changes in the flow and sediment regime of the river which will, in turn, cause possible alterations in the channel morphology. More specifically, the objective established for Workshop Problem Number 1 is to discuss the short and long term changes of the Yampa River to the following stream characteristics: 1) meander pattern, 2) configuration of the channel, 3) substrate material, and 4) pool-riffle sequence. The data supplied by the U.S. Fish and Wildlife Service include four small reaches of the river, only two of which are to be considered in any detail. They are 1) the Box Elder reach and 2) the Lily Park reach. The Lily Park reach is located just above the Little Snake River and the Box Elder reach is located just above the junction of the Yampa River with the Green River.

DATA SUMMARY

Available Data

Available data supplied by the U.S. Fish and Wildlife Service include:

I. General Information
  1. Maps of Yampa River basin, showing locations of gaging and sediment monitoring stations, proposed reservoir locations, and existing channel geometry stations.
  2. Key to USGS gaging and sediment monitoring stations.
  3. Description to physiography, geology, and precipitation characteristics of Yampa basin.
  4. Present and potential sediment yields in the Yampa River basin by E. D. Andrews, U.S. Geological Survey.

II. Data for Yampa River near Maybell, Colorado
  1. Present cross-sectional profiles for Maybell reach.
  2. Streamflow data.
    A. Average monthly discharges from 1910 to 1976.
    B. Projected average monthly discharges at Maybell, with Juniper reservoir.
    C. Peak flow data.
      1. Peak flows for period from 1904 to 1962.
      2. Peak flows, 1904-1978, with Weibull plotting positions and annual exceedence probability.
    D. Average monthly discharges, October 1975 to September 1978.
    E. Snowmelt hydrographs for 1976, 1977 and 1978.
  3. Sediment load data for Maybell reach.
    A. Suspended sediment loads and particle size analyses, Yampa River near Maybell, for the period November 1950 to September 1957.
    B. Suspended sediment discharge, Yampa River near Maybell, for the water year October 1977 to September 1978.
    C. Miscellaneous sediment discharge data for stations upstream from Maybell, for the water year October 1977 to September 1978.
      1. Williams Fork at mouth (2497.5).
      2. Yampa River below Craig (2476).
      3. Wilson Creek near Axial, CO (2506).
      4. Stokes Gulch near Hayden, CO (2444.7).
      5. Yampa River below Elkhead Creek (2465.5).
    D. Particle size analysis of bed material, Yampa River near Maybell.
  4. Area - elevation and capacity - elevation tables for Juniper Reservoir.

III. Data for Lily Park reach, Yampa River.
  1. Map of study area.
  2. Present cross sections measured at a discharge of 800 cfs.
  3. Present cross sections measured at a discharge of 5000 cfs.
    A. Average monthly discharges at Lily, for the period 1910 to 1976.
    B. Projected average monthly discharges at Lily, with Juniper and Cross Mountain.

C. Projected average monthly discharges, Little Snake River at mouth, with Juniper - Cross Mountain.
D. Peak flows, Little Snake River.

4. Suspended sediment discharge, Little Snake River at Lily, October 1977 to September 1978.
5. Area and capacity data for Cross Mountain Reservoir.

IV. Cross-sectional data for the Yampa River at Mantle Ranch.

V. Cross-sectional data for the Yampa River at Box Elder Campground.

Topography

Figure 1 gives a general location map of the study area. U.S. Geological Survey maps were utilized to approximate the channel bed profile. Figure 2 indicates that the bed slope of the study area changes from 2 ft/mile to 80 ft/mile (4×10^{-4} to 152×10^{-4}). The steepest reach is near the Cross Mountain Canyon where the Cross Mountain Dam is proposed. Variation of bed slope with river distance is given in Figure 3. It is important to note that the bed slopes of Maybell reach, Lily Park reach, and Box Elder reach are comparable. Excluding the special geological condition near the Cross Mountain Canyon area, the channel bed slope steepens below the confluence with the Little Snake River, then the bed slope flattens in the downstream direction. The steepening of the slope below the Little Snake River may be due to significant sediment inflow from the Little Snake River.

Cross-sectional Data

Available cross-sectional data cover only four isolated and short reaches. These short reaches are Maybell, Lily Park, Mantle Ranch, and Box Elder. Representative cross sections for these four reaches are given in Figures 4 to 7. Average top widths for approximately bank full flows for these four reaches are 280, 409, 200, and 379 ft respectively. The cross sections are practically unchanged for the range of measured flow conditions. Available cross-sectional data may be enough to represent the subject reaches for study of fish habitat. However, they are not sufficient to conduct any quantitative evaluation of river response.

Proposed Reservoirs

Elevation-capacity curves for the two proposed reservoirs are available. Only the proposed Cross Mountain Reservoir will be considered in the analysis. The main purpose of the proposed reservoir is for flood control. This capacity would provide a significant reduction of flood peaks. Information related to reservoir operation is unavailable.

Hydrology

Streamflow records of the Yampa River near Maybell, Colorado (USGS Station No. 09251000) and Little Snake River near Lily, Colorado (USGS Station No. 09260000) are available. In addition, the projected monthly

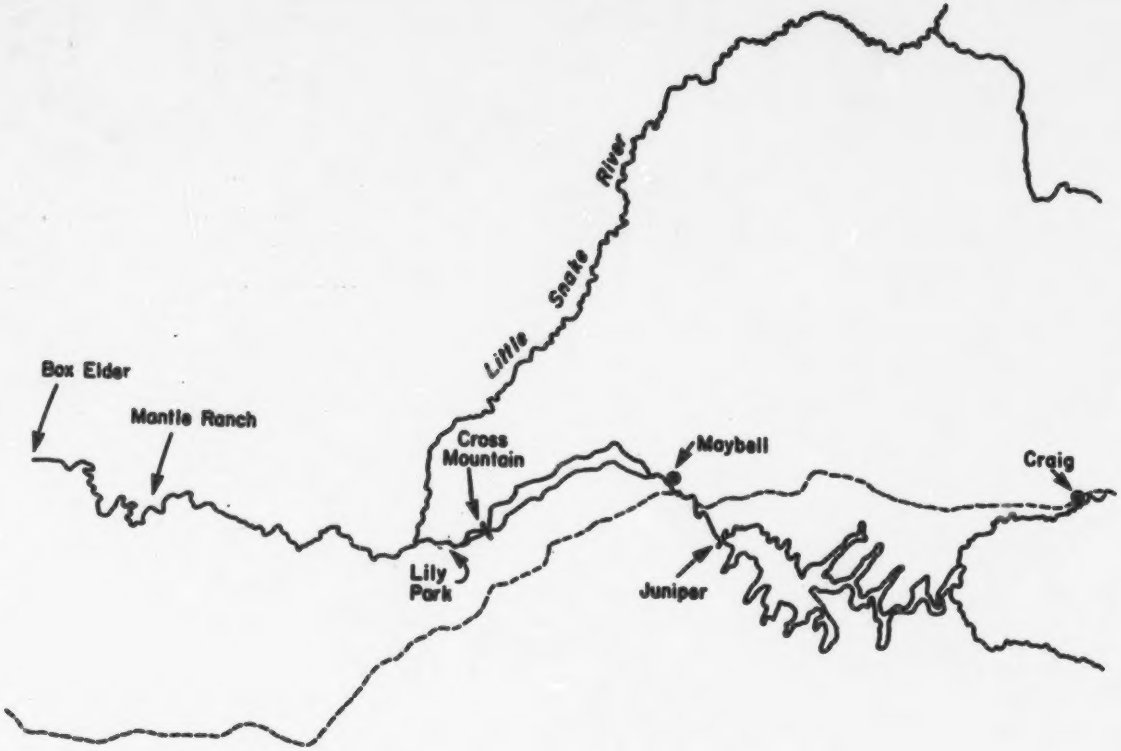

Figure 1. General location map.

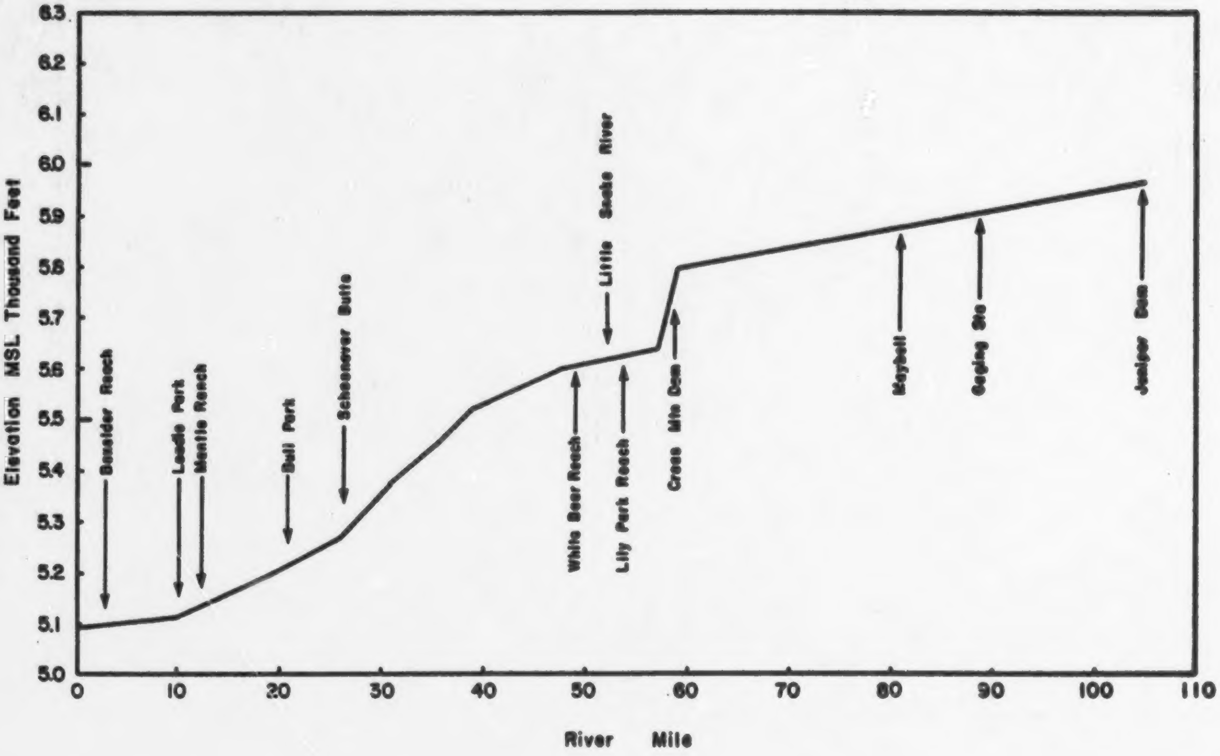

Figure 2. Yampa River bed profile.

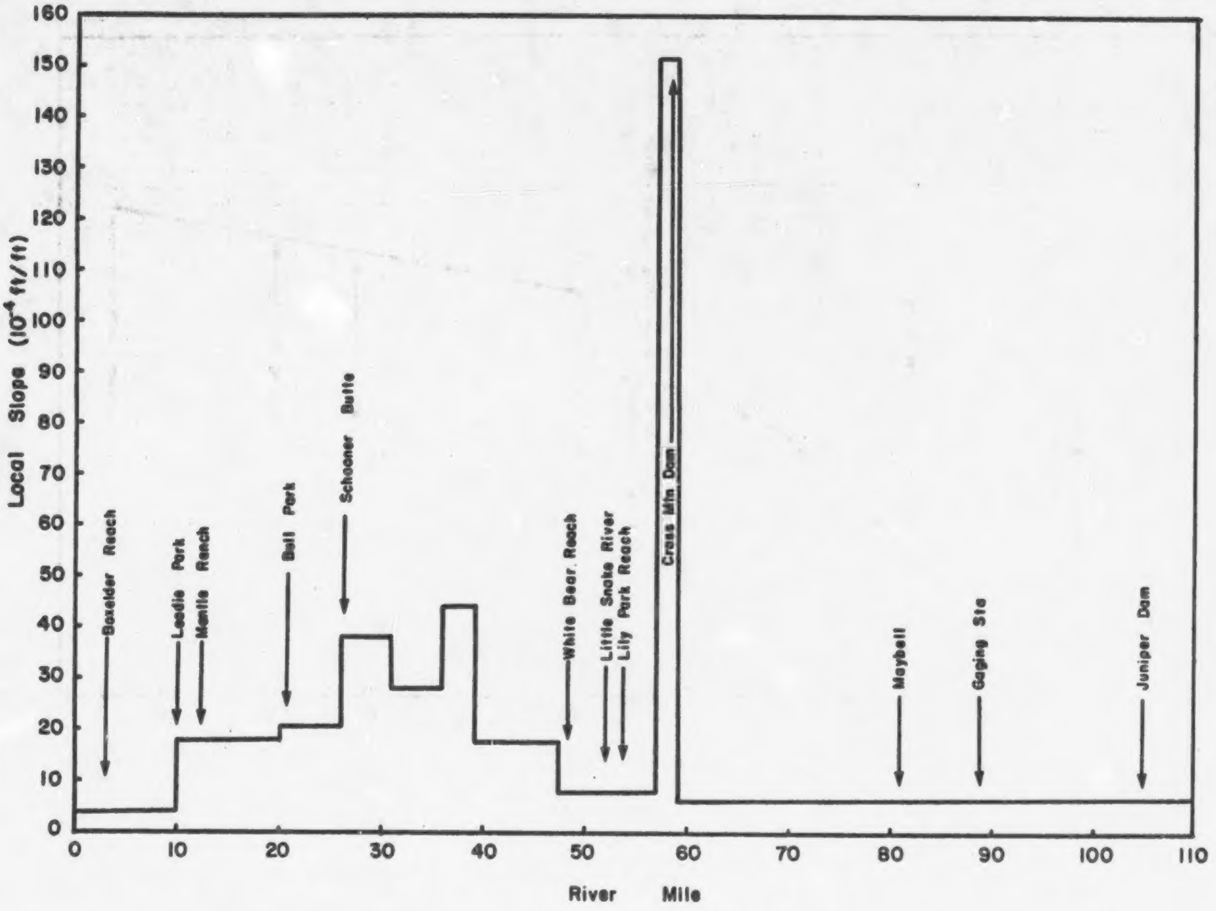

Figure 3. Variation of slope with river distance.

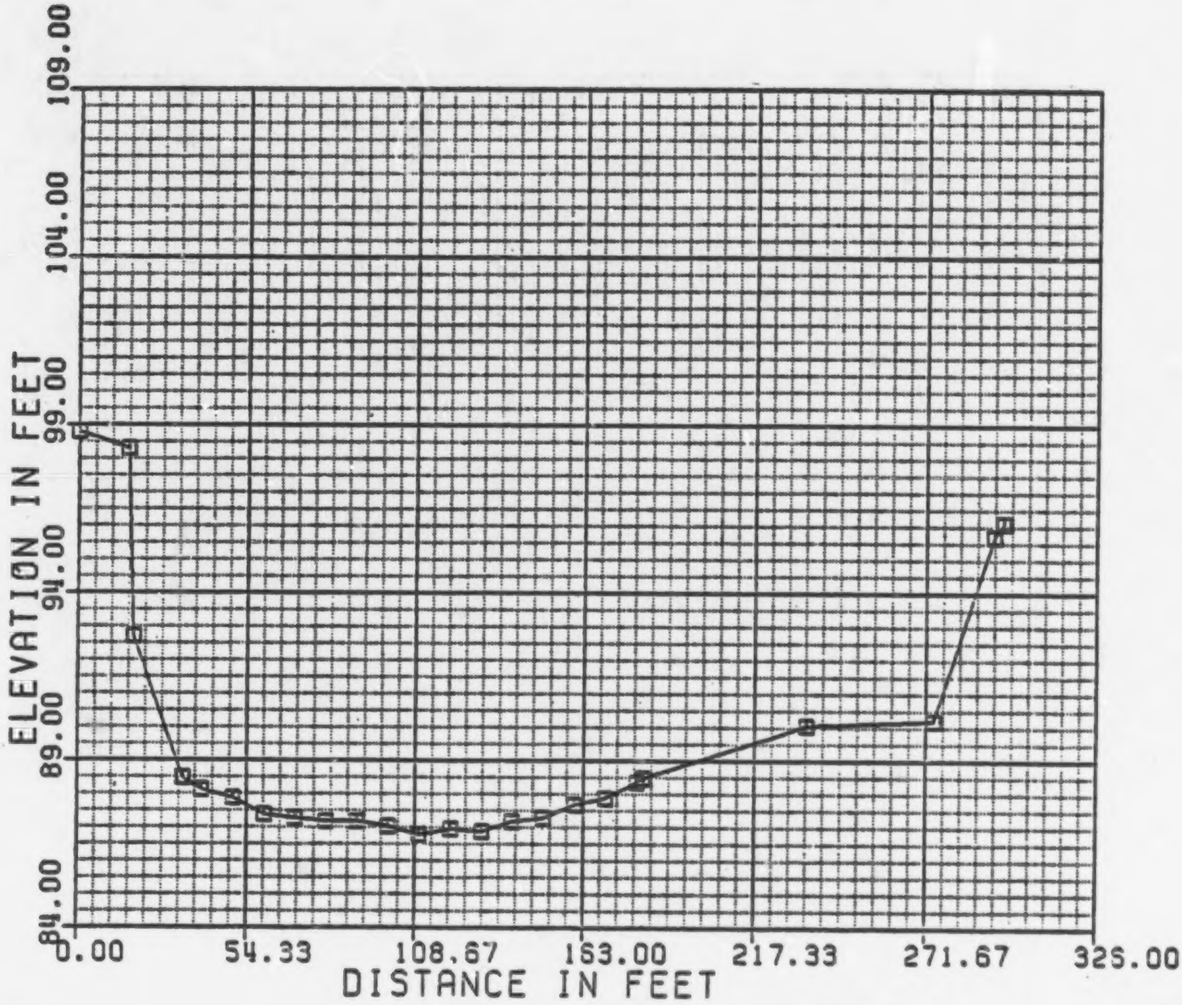

Figure 4. Maybell reach cross section 21.

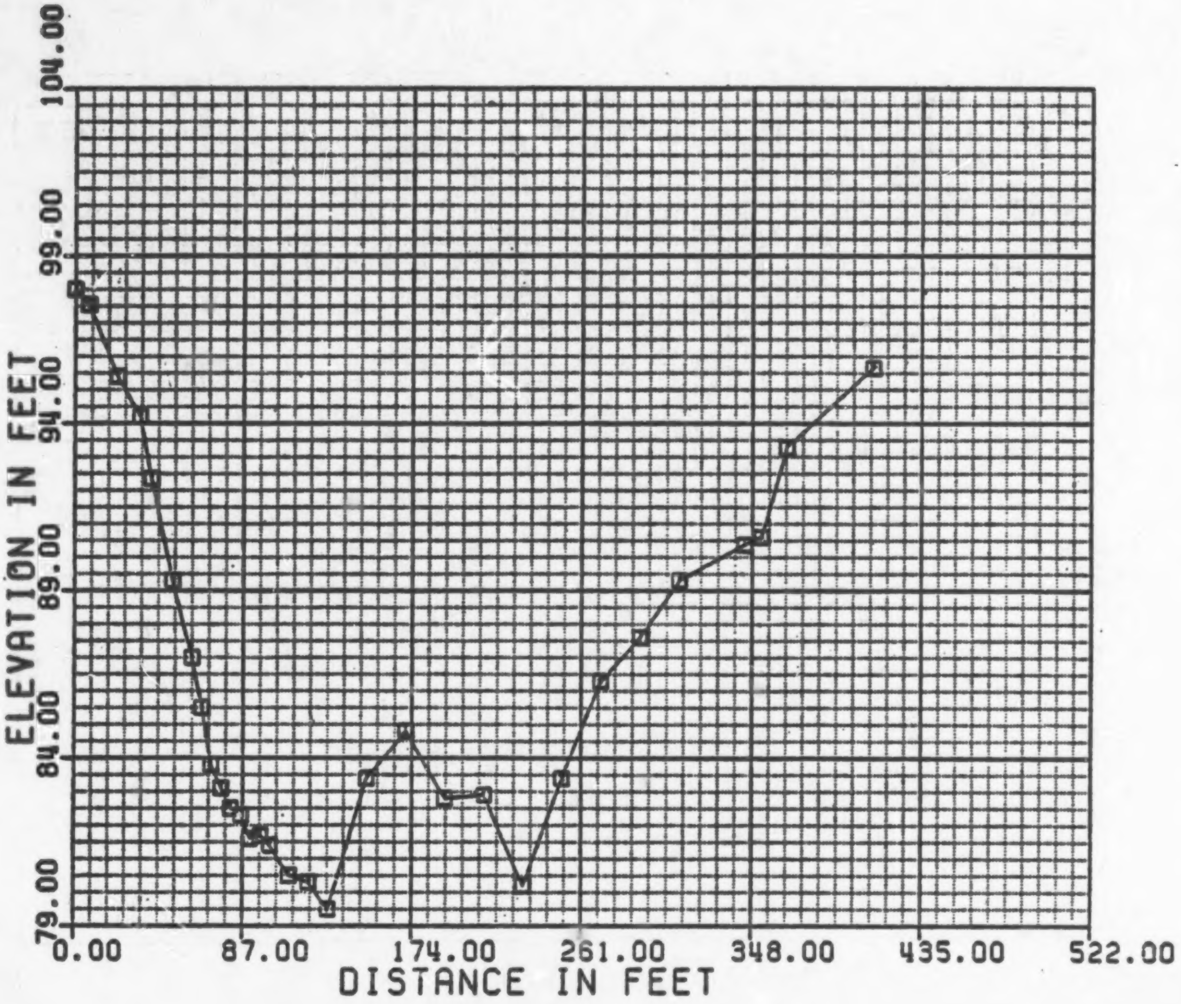

Figure 5. Yampa River at Lily Park, cross section 338.

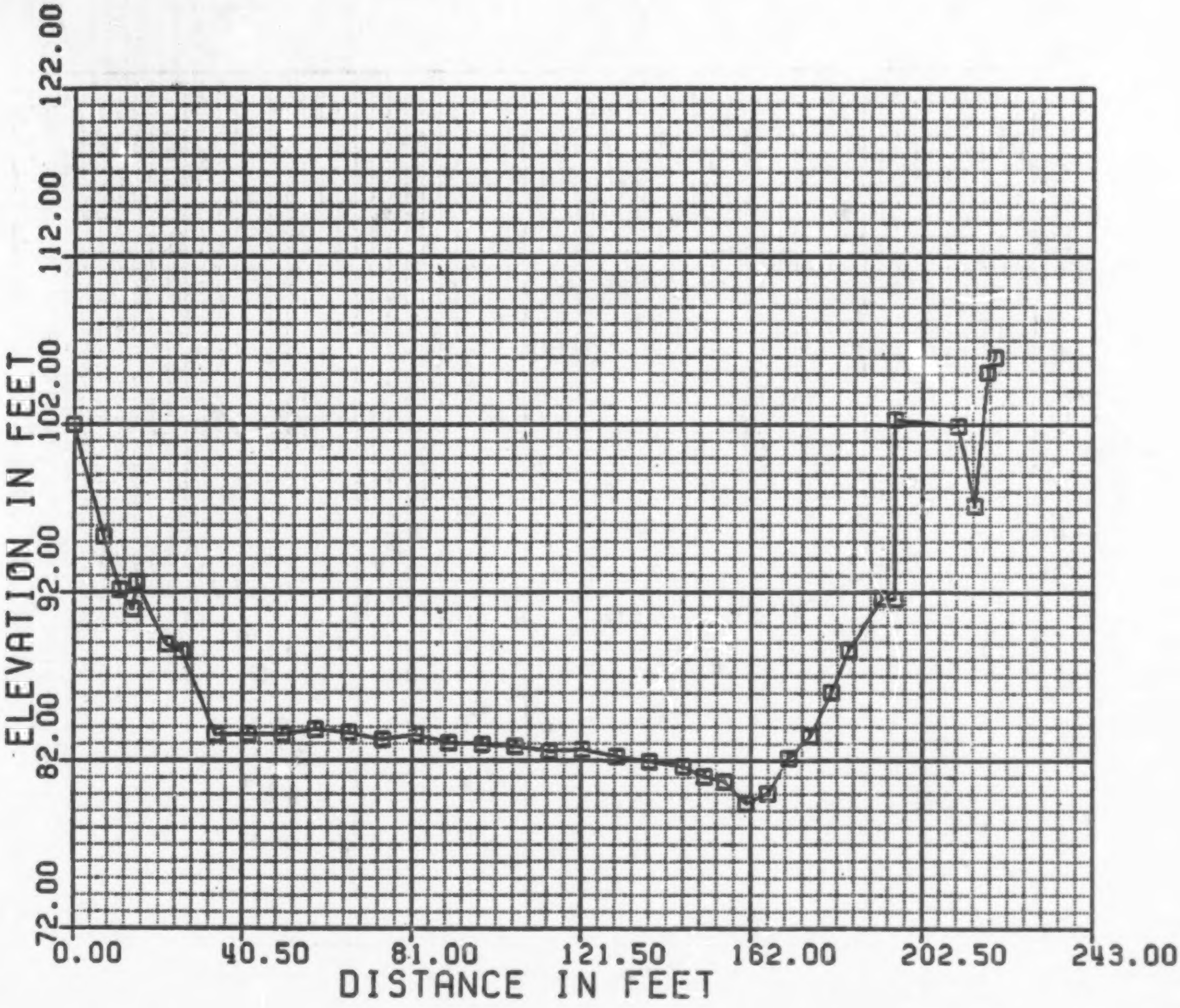

Figure 6. Dinosaur National Monument near Mantle Ranch, cross section 150.

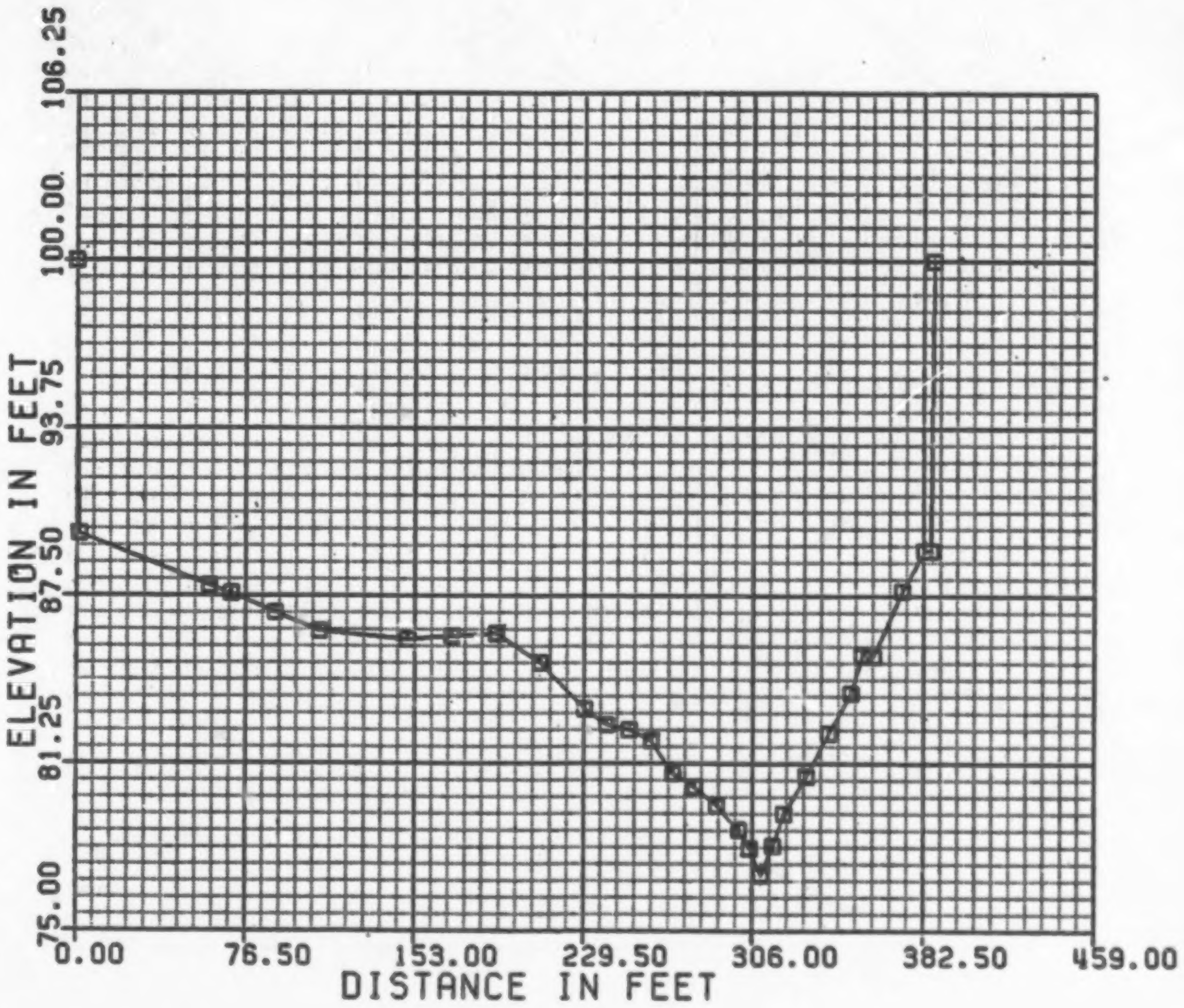

Figure 7. Dinosaur National Monument, Box Elder reach, cross section 24.

streamflows considering the Cross Mountain Reservoir are given. However, effects of the proposed reservoir on reducing the flood peaks are not provided. The proposed Cross Mountain Reservoir will reduce some of the annual streamflows and the reduction of flood peak will be more significant. Figure 8 gives the annual average daily discharge of the Yampa River near Maybell with the effect of the Cross Mountain Reservoir and the Little Snake River near Lily. It is important to note that flow released from the Cross Mountain Reservoir is on the average only about 1.5 larger than the flow from the Snake River. Furthermore, the Snake River will not be regulated and will probably produce higher flood peaks than those released from the Cross Mountain Reservoir.

Figures 9 and 10 show the flood frequency analysis of the flows at stations of the Yampa River near Maybell and the Little Snake River near Lily. No information is available about the flood frequency of the Yampa River modified by the operation of the Cross Mountain Reservoir.

The Yampa River has a strong seasonal discharge trend, dominated by snowmelt runoff during spring and early summer (Figure 11). Many high peaks were produced by events involving rain on snow pack. Many measured stage-discharge records are available. Records show that the stages for 4,000, 10,000 and 18,000 cfs are approximately 4, 7, and 10 ft, respectively.

Bed and Bank Material

A single bed material sample is available for the Yampa River near Maybell, Colorado. The size distribution curve is given in Figure 12. Personal communication with John Andrew indicates that the bed material size distributions from Maybell to Lily Park reach upstream of the confluence with the Little Snake River are similar. This reach is essentially a gravel-cobble bed stream. With high sediment load from the Little Snake River, the Yampa River below the confluence with the Little Snake is predominantly a sand bed stream. Extensive bed and bank material samples would be required to align the data to describe the "as is" condition. Exploratory drill results if available would also be helpful.

Sediment Transport

Long term suspended sediment data are available for the upper reach and for the Little Snake River. According to E. D. Andrews of the U.S. Geological Survey, the lower Little Snake River sub-basin contributes about 60% of the total Yampa River basin sediment yield, although it represents less than 35% of the area and supplies less than 3% of the streamflow. However, it is suspected that most of the sediment from the Little Snake River is fine sediment in the range of silt, clay and fine sand.

Examination of the available suspended sediment data indicates that there is no correlation between measured suspended concentration and discharge. Concentration ranges from 40 to 1500 ppm, and the predominant size of suspended sediment is silt and clay. More sediment transport investigation is required.

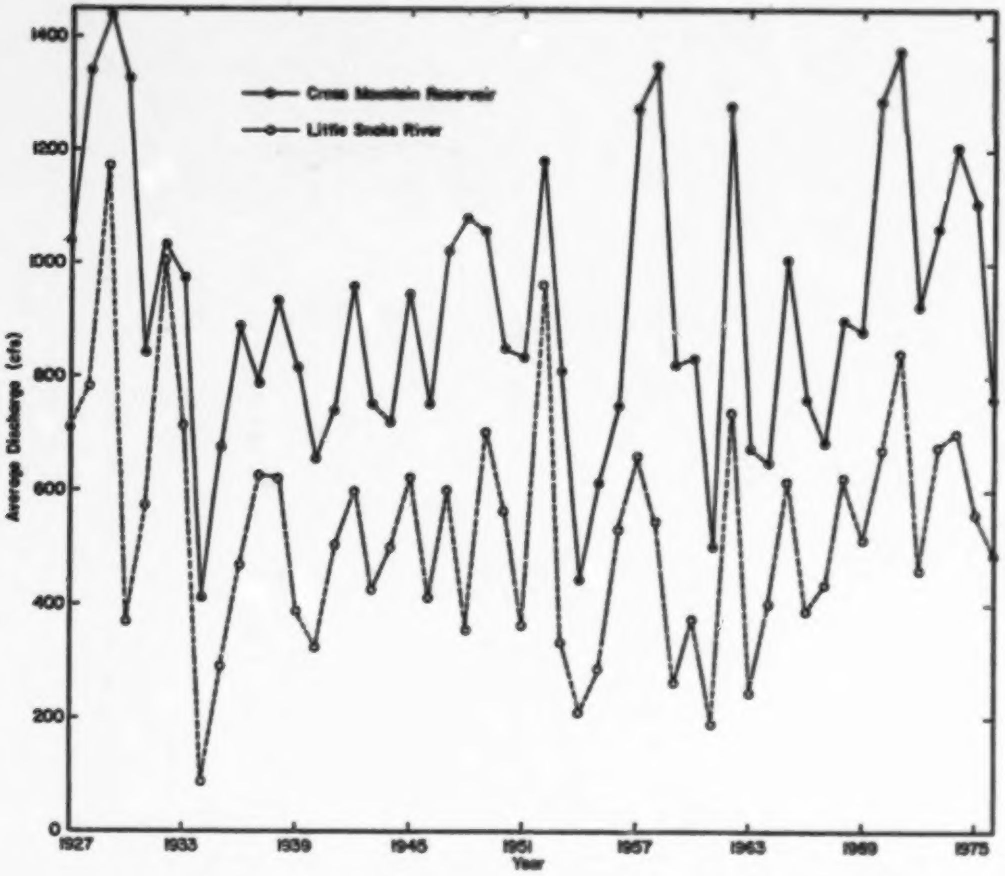

Figure 8. Comparison of flow from Cross Mountain Reservoir release and the Little Snake River.

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                                         FOLLOWING WRC GUIDELINES BULL. 17-A.           RUN-DATE  5/13/80 AT  134   SEQ  1.0001

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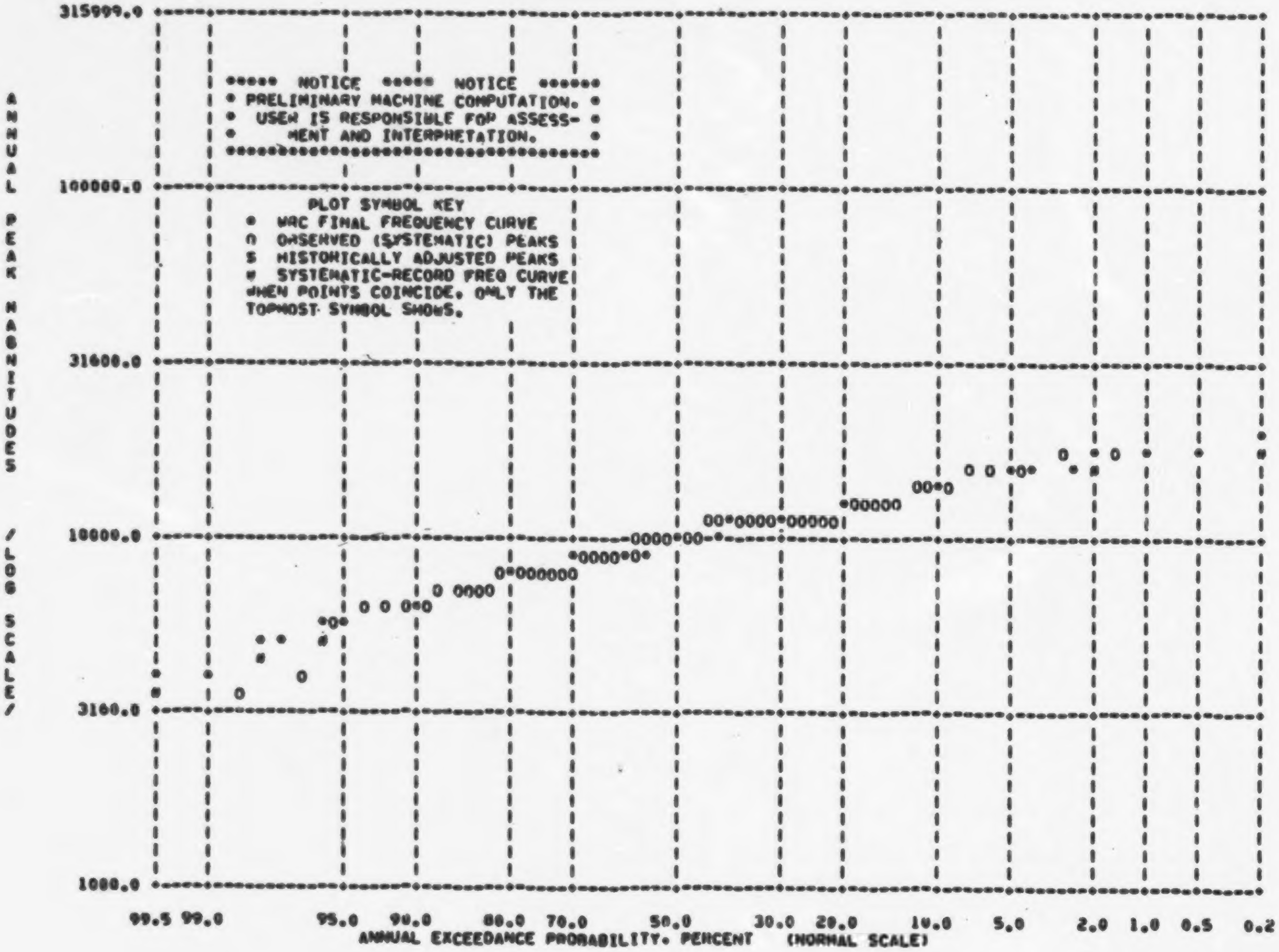

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  S  HISTORICALLY ADJUSTED PEAKS
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                              FOLLOWING WRC GUIDELINES BULL. 17-A.            RUN-DATE  5/13/80 AT  134   SEQ  1.0002

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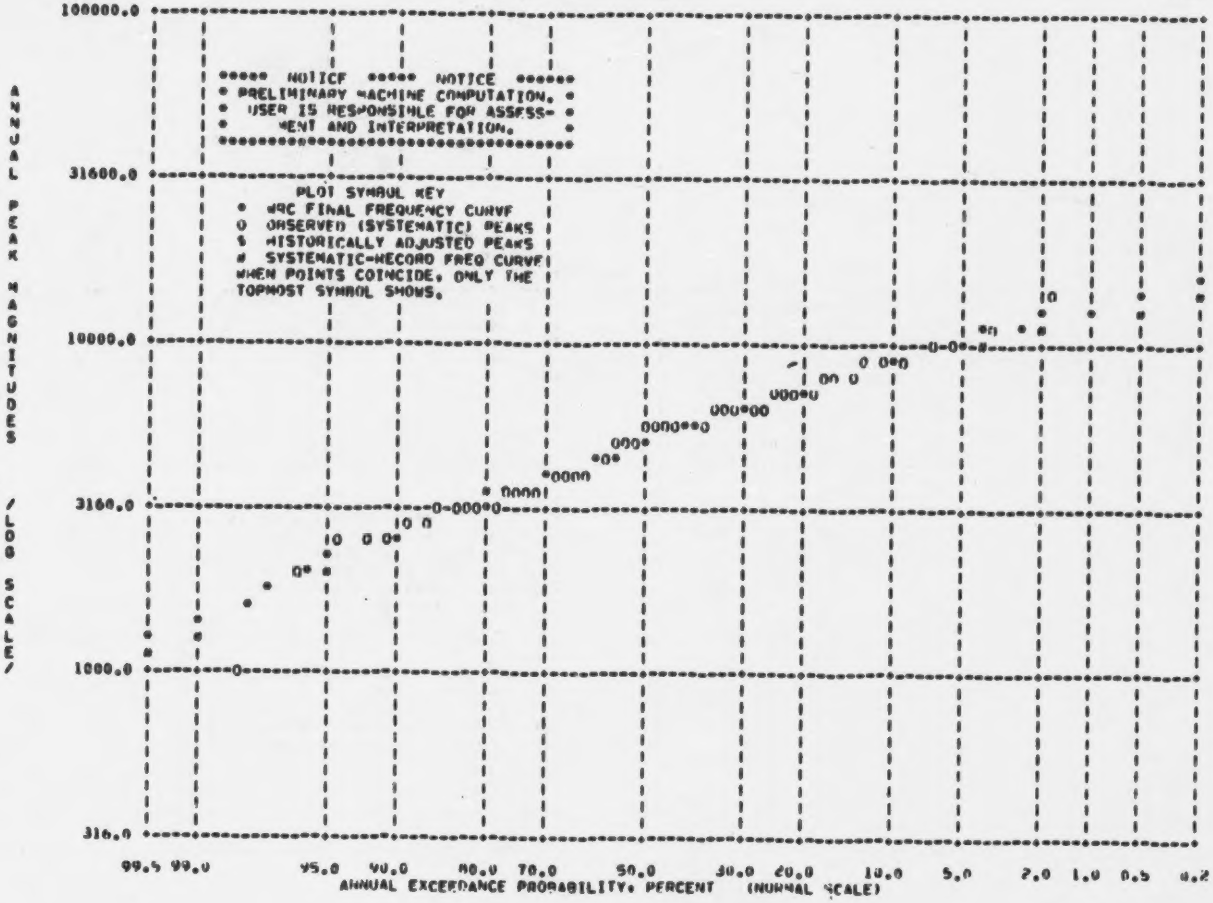

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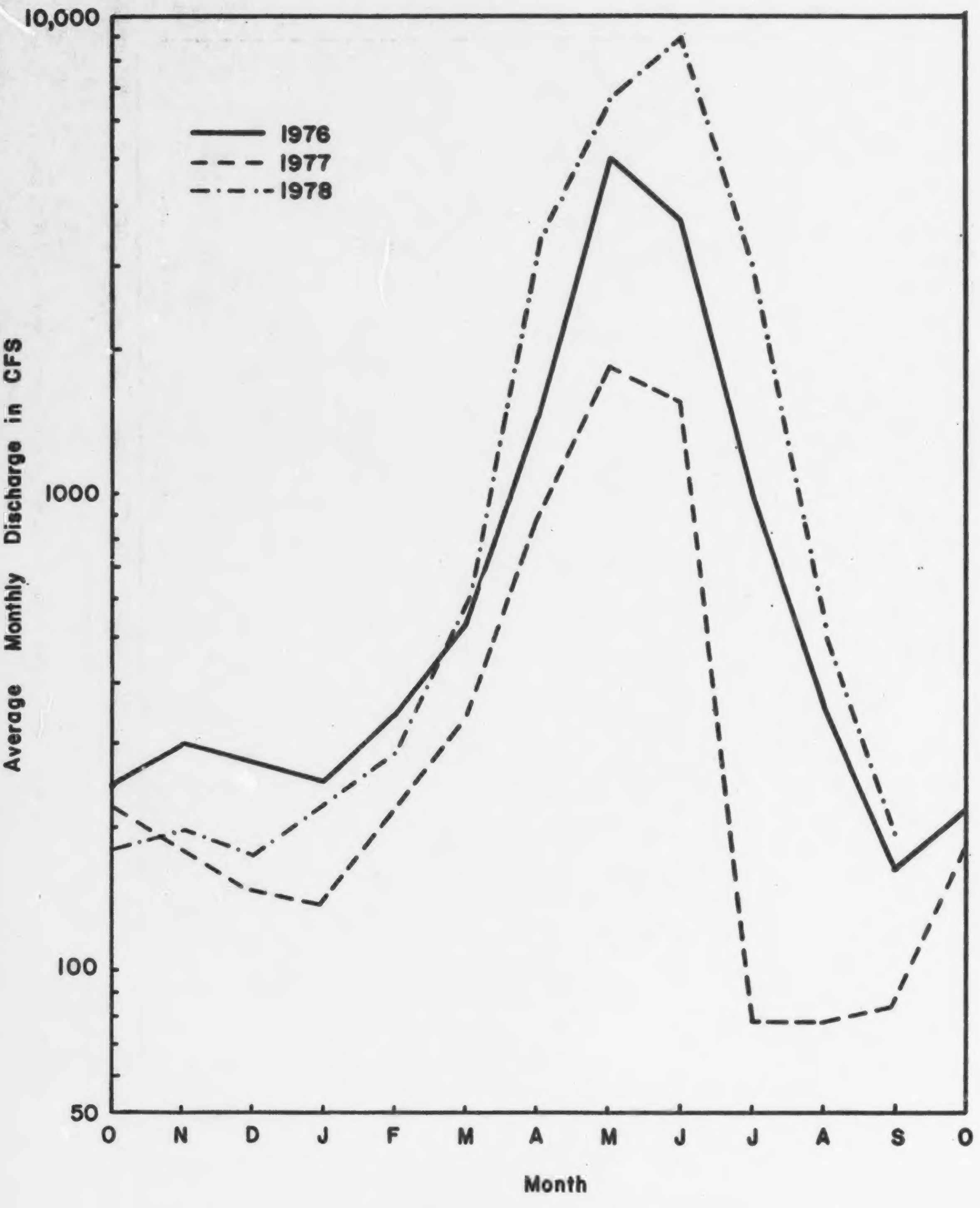

Figure 11. Monthly variation of flow on the Yampa River.

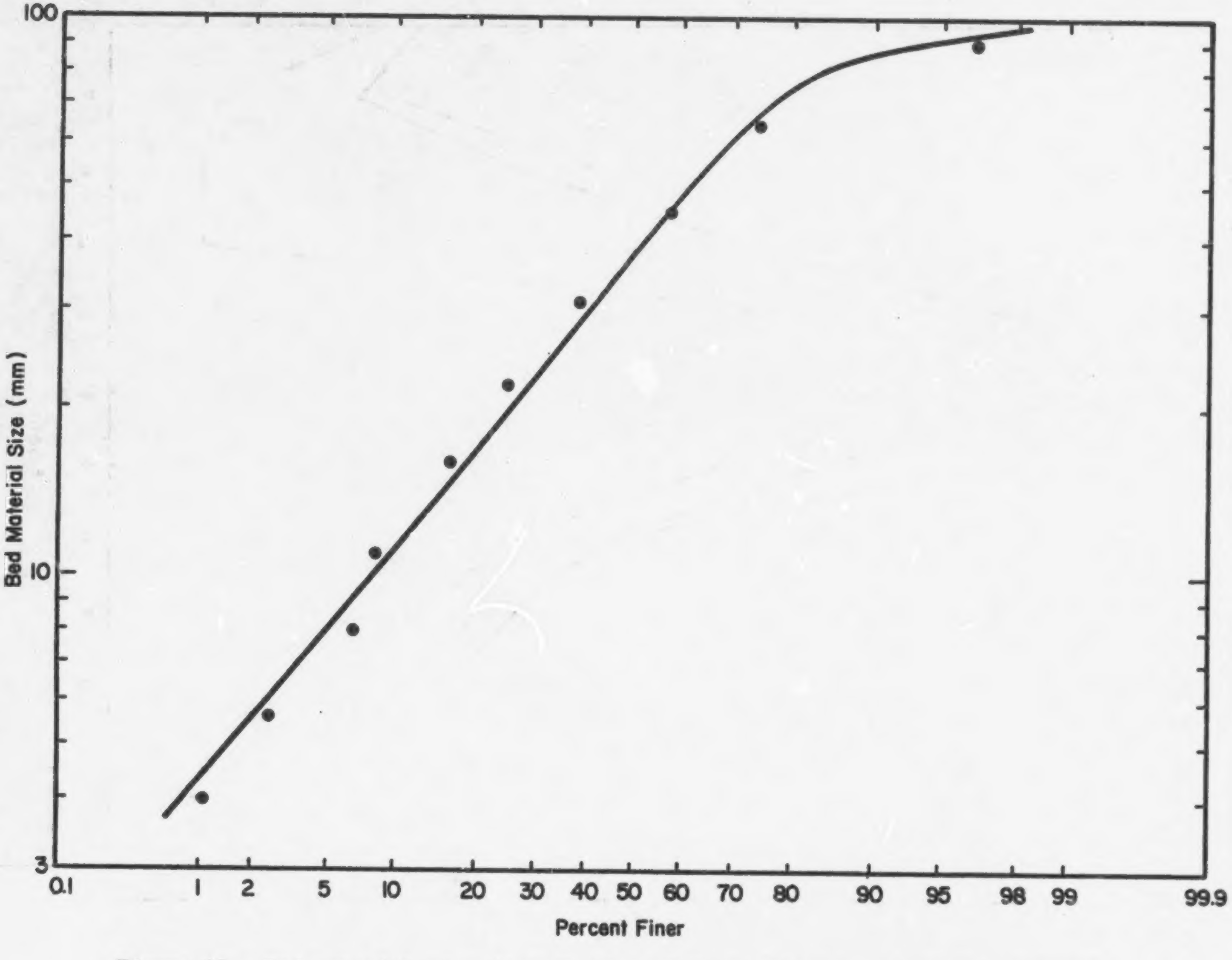

Figure 12. Bed material size distribution of Yampa River near Maybell, Colorado.

Morphology

Time-lapsed aerial photographs are not available. With only topographic mapping at 1:62500, little can be derived for the channel morphology. Personal communication with John Andrew reveals that the Yampa River may be in the transition regime. Parts of the river are braided and others are meandering.

ANALYSIS

General Approach

There are three basic levels of approach for predicting the downstream morphology changes: 1) qualitative involving geomorphic concepts, 2) quantitative involving geomorphic concepts and basic engineering relationships, and 3) quantitative involving sophisticated mathematical modeling concepts. A mathematical model is simply a quantitative expression of a physical process or phenomenon. The physical processes governing watershed and river responses are very complicated. Figure 13 illustrates some key elements in the mathematical modeling of watershed and river responses.

A qualitative analysis can provide insight and direction to an analysis of complicated river response problems. Investigation of complex problems with many variables usually requires a massive amount of data. Application of a strict quantitative analysis under these conditions results in many tables and charts summarizing the results of many calculations. Understanding and applying these results to the problem solution is extremely difficult. A qualitative analysis applied before or during the quantitative effort indicates those variables and relationships that are actually significant to the given problem. Furthermore, the qualitative analysis will aid in the selection of the level of quantitative analysis.

Available data on the workshop problem are not sufficient to obtain a conclusive answer. Only a qualitative statement can be obtained based on the data that are currently available.

Qualitative Analysis

Based on the hydrology and bed slope information, it can be determined that the Yampa River is in the transition regime. This determination was based on the work of Lane (Simons and Senturk, 1977) by noting that the product of $SQ^{1/4}$ (bed slope and one-fourth power of dominant discharge) is 0.006 for the Yampa River. The transition regime (or intermediate stream) is capable of braiding or meandering or both depending on the local condition.

Qualitative geomorphic analysis of morphology changes is based on the concept of equilibrium. The qualitative approach assumes that rivers strive, in the long run, to achieve a balance between the product

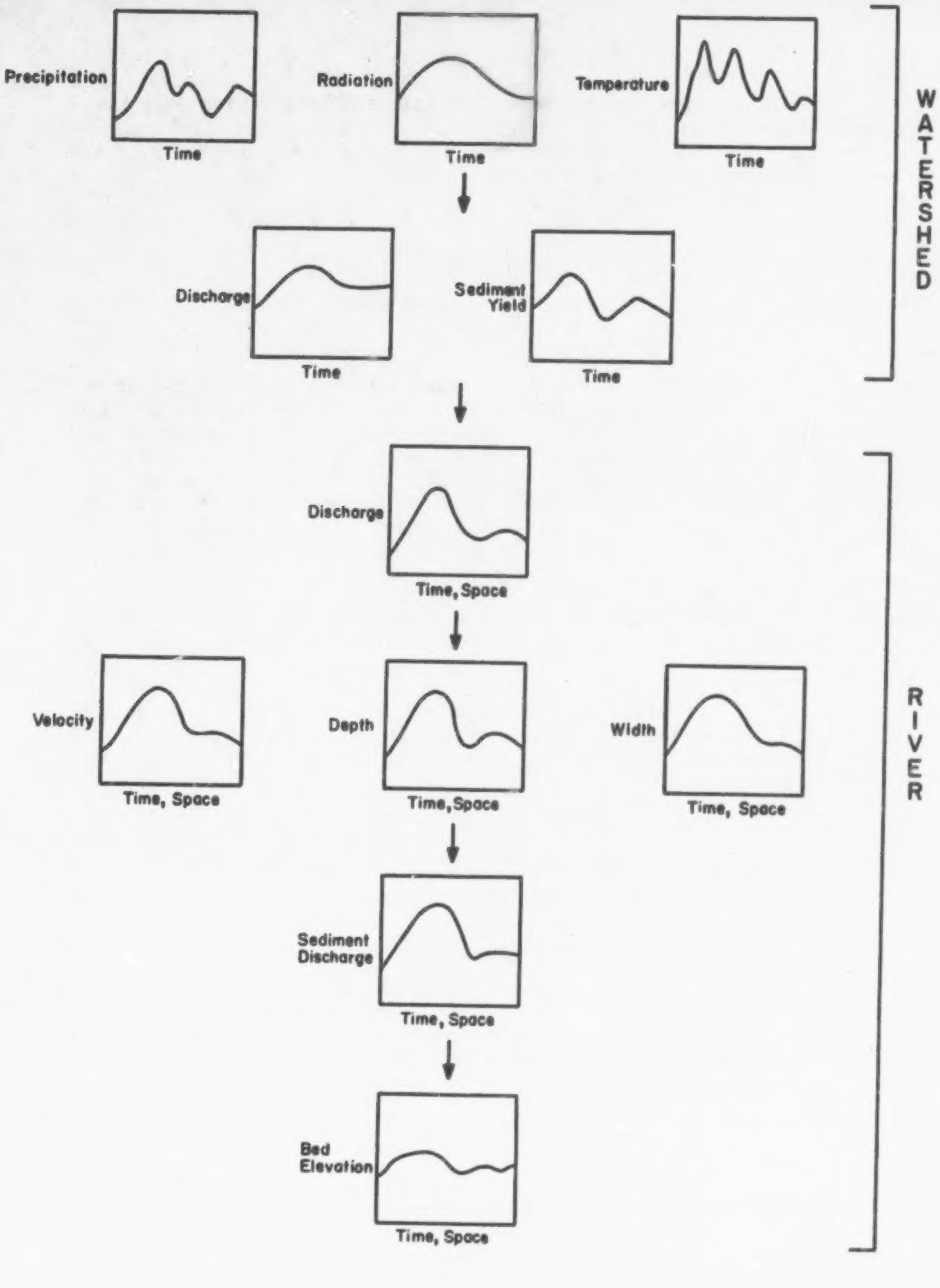

Figure 13. Key elements in the watershed and river analysis.

of water flow and channel slope and the product of sediment discharge and size. The most widely known geomorphic relation embodying the equilibrium concept is known as Lane's principle. The Lane relation is

$$Q_s \, d_{50} = Q_w \, S \tag{1}$$

where Q_s is sediment discharge, d_{50} is particle size, Q_w is water discharge, and S is bed slope.

Application of the relation in Equation 1 to the workshop problem will indicate that there is a potential for degradation in the Lily Park reach if the reach is not armored due to the clear water release from the proposed Cross Mountain Reservoir. Or

$$Q_s^- \, d_{50}^0 \propto Q_w^0 \, S^- \tag{2}$$

where S^- indicates the potential for degradation. However, an examination of the bed material in the Maybell reach indicates that there is a strong possibility for an armoring effect in the Lily Park reach, and therefore the degradation may be minimal. The armored material in the major bars and riffles probably moves only during large flows and it may never move under regulated flow conditions. Only some of the fine sediment will be gradually cleaned out of the system. Furthermore, assuming that the bank material is either armored or consists of bed rock, lateral migration would also be curtailed. Therefore, the morphology of the Yampa River in the Lily Park reach will likely be unaffected by the construction of the Cross Mountain Reservoir. The meander pattern, channel configuration, and pool-riffle sequence will be essentially unchanged. The substrate material will become slightly coarser and will not affect fish and other aquatic biota significantly. The change in the flow regime and possibly the temperature regime may have some effect.

Below the Little Snake River, the Yampa River may experience aggradation due to the large sediment load of the Little Snake River and the reduced capacity of the Yampa River due to the construction of the dam. This can be concluded from the Lane relation as follows.

$$Q_s^0 \, d_{50}^0 \propto Q_w^- \, S^+ \tag{3}$$

Box Elder reach is in the lower reach and most likely will experience aggradation. This aggradation process will steepen the slope and may alter the stream regime from the transition to the braided stream. Current bed material in the Box Elder reach is predominantly from the Little Snake River and the substrate material is likely to remain the same for the post-project condition. Meander pattern, channel configuration, and pool-riffle sequence will likely be altered significantly. The stream is likely to become braided with more pronounced multiple channels. Width-depth ratio will tend to increase and the channel will be more unstable. Due to the confinement of the bluff lines existing on both banks, the braided channel will be confined in width from bluff line to bluff line. Degree of alteration is dependent on the extent of changes in sediment supply, hydrograph and others. Only a quantitative approach can provide a meaningful answer, but the qualitative analysis indicates that this level of effort should be made in the Box Elder reach.

Quantitative Geomorphic Analysis

As mentioned earlier in the qualitative analysis, the key to predicting the potential change in the Lily Park reach is an examination of the armoring effect. Utilizing the Shields criteria coupled with the Manning equation, an analysis of incipient motion was conducted for the Lily Park reach. Figure 14 gives the relation between incipient particle size and discharge. For a one percent flow (or 100-year flood), discharge is approximately 20,000 cfs. Average particle size that will be in motion is about 21 mm. According to Figure 12, material of this size corresponds to the size that is 30% finer by weight. This implies that on the average, 70% of the particles will not be in motion during a 100-year flood. This supports the assumption of armoring effect made in the qualitative analysis. Due to the unavailability of the bed material size distribution in the Lily Park reach, the above conclusion is at best semiquantitative.

Equilibrium bed slope in the lower reach below the Little Snake River confluence can be approximated by knowing sediment supply rate, flow rate, the sediment transport equation, and considering any manmade or natural control points. A procedure for estimating equilibrium bed slope is outlined below.

Equations of sediment transport and Manning's equation applied to a unit width of channel form a basis for the analysis. The sediment transport equation (bed load) can be written as

$$q_b = a\, V^b\, D^c \tag{4}$$

where q_b is the unit width bed material transport rate; a, b, and c are constants at each site, V is mean velocity, and D is hydraulic depth. The unit width Manning's equation and the flow continuity equation are written as

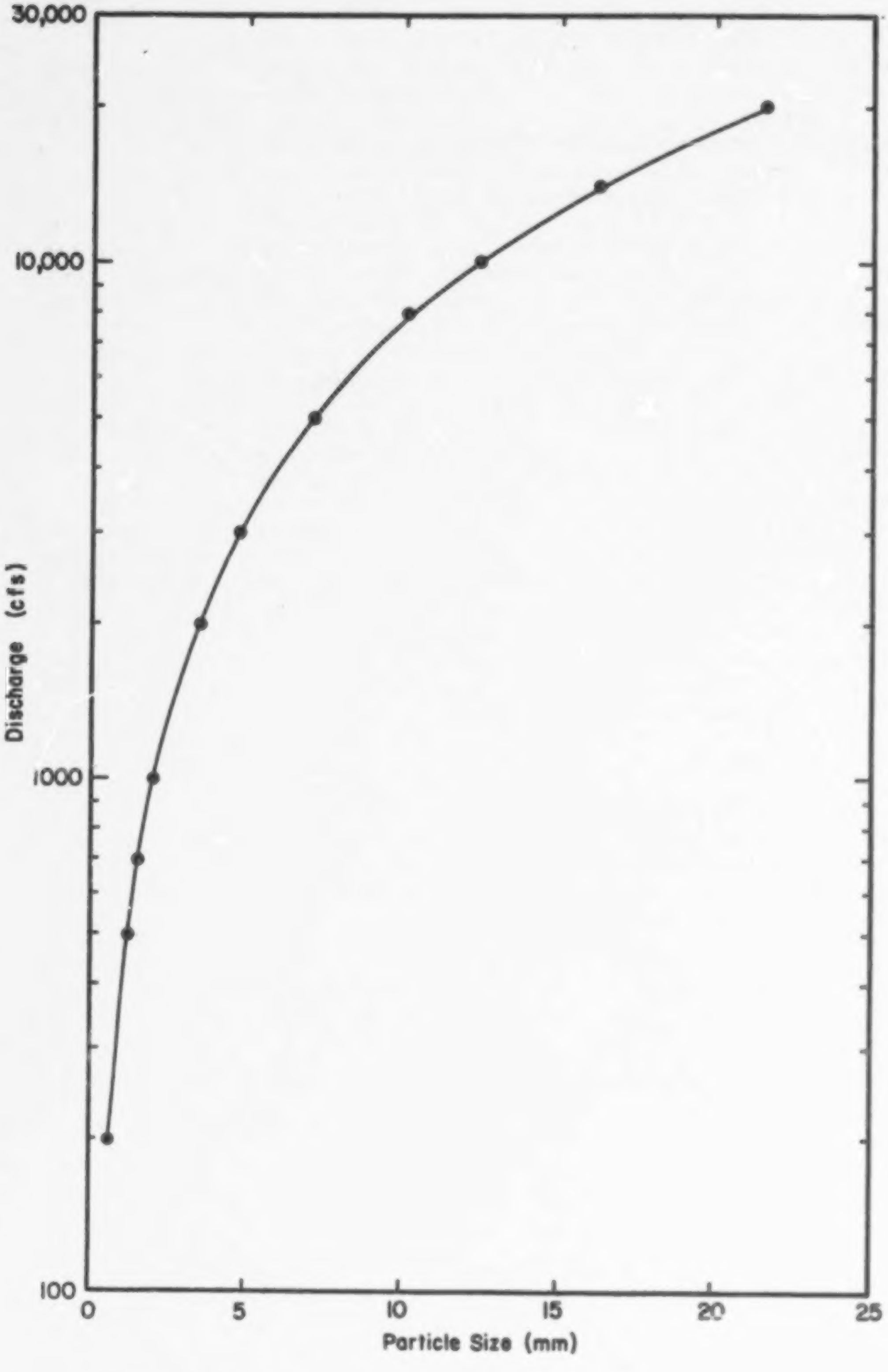

Figure 14. Incipient motion analysis for the Lily Park reach.

$$q = \frac{1.49}{n} D^{5/3} S^{1/2} \qquad (5)$$

and

$$q = VD \qquad (6)$$

where q is the unit width discharge, n is Manning's roughness coefficient, D is depth of flow, and S is slope.

In simultaneous solution with the estimated sediment inflow rate, the new bed slope can be determined by the following procedure. For a selected design flow, this unit width sediment supply rate is set equal to Equation 4. This equation is a function of V and D along with the unit width water continuity equation. These two equations are solved simultaneously for V and D. Computed depth is then substituted into the unit width Manning's equation in order to solve for the new equilibrium slope. This new slope can be compared with bed slope or another computed slope under other flow conditions to indicate possible river response under various conditions.

The above procedure is only for approximations and is presented for illustration of the steps involved. For a more accurate analysis, channel geometry equations established using cross-sectional data should be utilized.

Mathematical Modeling

It is recommended that the dynamic nature of the Box Elder reach be studied by applying the mathematical modeling technique involving the elements outlined in Figure 13. Due to the short study reach involved, a known discharge sediment routing method is recommended. Because of the heterogeneous distribution of the bed material, a procedure for routing sediment by size fraction should be utilized. As outlined in Figure 13, the procedure involves estimation of sediment inflow from watersheds. Transporting capacity of each reach is determined utilizing the hydraulic conditions that can be determined by a backwater profile computation. The sediment routing procedure is accomplished by applying the sediment continuity equation and considering the size distribution of the upstream sediment supply and the bed material. The unsteady flow nature of the problem can be approximated by a series of semi-steady flow conditions.

CONCLUSIONS

This paper reviews the data needs, suggests methods of analysis, and presents a qualitative assessment of downstream river morphological changes from the proposed reservoir construction on the Yampa River, Colorado.

Available data are not sufficient to obtain a conclusive answer; therefore, only a qualitative statement can be obtained. It is imperative that a site visit be conducted before attempting to draw any conclusions related to the analysis of river response. Time-lapse aerial photographs are often useful in identifying river behavior. Based on the available data, qualitative conclusions regarding possible response of the downstream channel follow.

Morphology of the Yampa River in the Lily Park reach upstream of the confluence of the Little Snake River will likely be unaffected by the construction of the Cross Mountain Reservoir. The meander pattern, channel configuration, and pool-riffle sequence will be essentially unchanged. Substrate material will become slightly coarser and will not affect aquatic biota significantly.

Below the Little Snake River, the Yampa River may experience aggradation due to the large sediment load introduced by the Little Snake River and reduced transport capacity of the Yampa River due to construction of the dam. The Box Elder reach is in the lower reach and most likely will experience aggradation. This aggradation process will steepen the slope and may alter the stream regime from a transition to a braided stream. Thus, the meander pattern, channel configuration, and pool-riffle sequence will likely be altered significantly. Current bed material in the Box Elder reach is predominantly from the Little Snake River and the substrate material will largely remain the same for the post-project condition.

Additional data that are required to reach a more conclusive answer include:

1. Time-lapse aerial photographs,
2. Cross-sectional data for the entire study area,
3. Bed and bank material data covering the entire study area and the Little Snake River,
4. More detailed sediment transport data,
5. Reservoir operation data and related structural work, and
6. Modified flood peak frequency affected by construction of the dam.

Level of analysis chosen could be different for different study reaches. A qualitative and quantitative geomorphic analysis will be sufficient to analyze the Lily Park reach. A more detailed study is recommended involving use of mathematical modeling to analyze the more complex reach below the Little Snake River.

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Simons, D. B., and F. Senturk. 1977. Sediment Transport Technology, Water Resources Publications, Fort Collins, Colorado.

POSSIBLE EFFECTS OF CROSS MOUNTAIN AND JUNIPER RESERVOIRS ON LILY PARK AND BOX ELDER REACHES ON YAMPA RIVER, COLORADO

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INTRODUCTION

The Cross Mountain and Juniper Dams may be constructed in the future. The Colorado Squawfish designated as an endangered species in the Lily Park and Box Elder Reaches downstream from the two reservoirs may be adversely effected by these two dams. The purpose of this paper is to present a preliminary analysis based on available hydrologic data to evaluate the potential effects of the two dams on the Colorado Squaw Fish.

DESCRIPTION OF THE AREA

Figure 1 shows the location of this area. Lily Park Reach is located a) immediately above the junction between the Little Snake River and the Yampa River and b) immediately downstream from the Cross Mountain Dam. The distance between the Box Elder Reach and the Cross Mountain Dam is approximately 59 miles.

As indicated by Figure 1, two main U.S. Geological Survey gaging stations, the Maybell Station on the Yampa and the Lily Station on Little Snake River, are in this region.

GENERAL ANALYSIS

Before a detailed analysis of the hydrologic data, it would always be useful to inspect the field situation and obtain an overview of the morphology of the river. Since field investigation is not possible in this case, one must examine the morphology of the river from given data. Both the Yampa River and the Little Snake River are meandering rivers. However, the Yampa River below the junction of Little Snake and Yampa River appears to exhibit a braided pattern. This indicates that the flow there has not been able to carry out all the sediment carried into the reach. This is an extremely important point to be considered later in more detail.

The longitudinal bed slopes for different reaches of the Yampa River are given in Table 1.

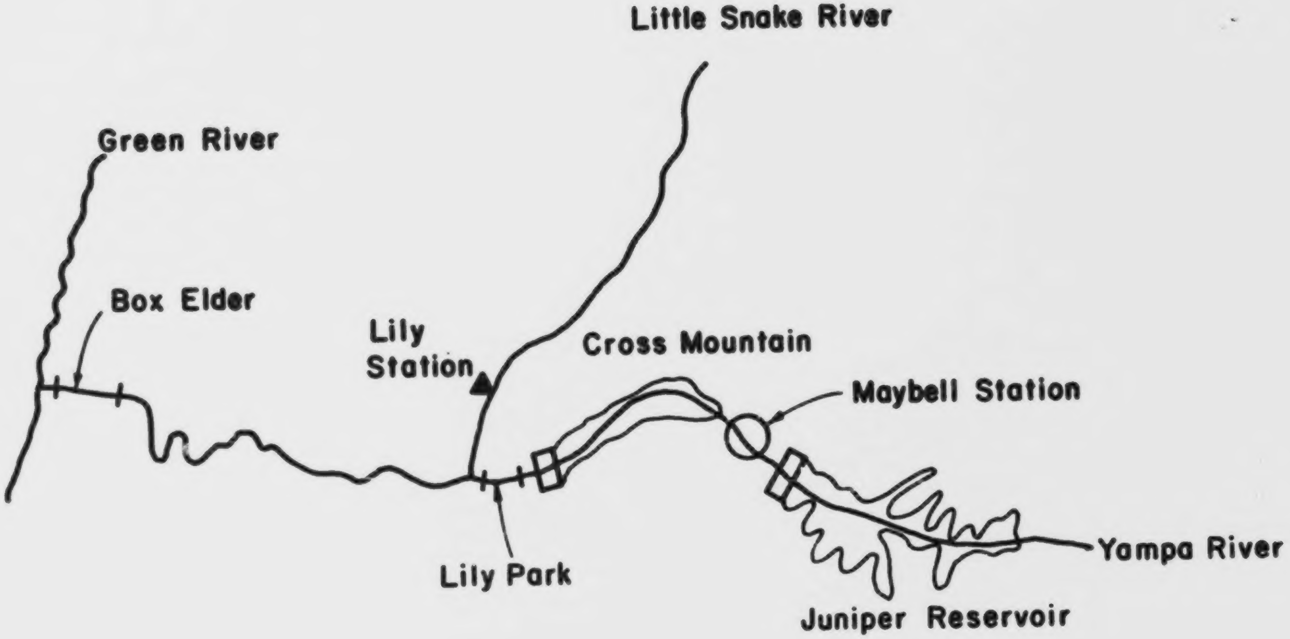

Figure 1. General location (not to scale).

Table 1. Longitudinal bed slopes for Yampa River downstream from Cross Mountain Dam (0 miles is at Box Elder).

Reach	Approximate Length (miles)	Bed Slope (ft/ft)
0 (Box Elder) to 20 miles	20	0.00085
20 miles to 38 miles	18	0.003
38 miles to Lily Park (56 miles)	18	0.0013
Lily Park (56 miles) to Cross Mountain Dam (59 miles)	3	0.01

The junction between Little Snake and Yampa River is at a distance of approximately 52 miles from Box Elder.

If both dams should be completed, the effects of Juniper Dam on both Box Elder and Lily Park Reaches are through the regulation of reservoir flow and potential erosion between the two dams. Both of these effects are minor; thus, it can be assumed that one needs to examine the river reach below Cross Mountain Dam.

As shown in Table 1, the variations of bed slope at different river reaches below Cross Mountain Dam are rather significant. These large differences indicate that the following two river reaches: a) river miles 20 to 38 and b) river miles 56 to 59 are rather stable with steep slopes. After the construction of the two dams, the river peak flows would be reduced and thus it is reasonable to assume that these two reaches would be stable in the future. The most critical reach to be examined should be the reach between river mile 38 to Lily Park. This is the river reach that the Little Snake River enters and that also exhibits a braided pattern.

HYDROLOGIC ANALYSIS

Reservoir Releases

The closest station on the Yampa River is at Maybell and flow data for this station provided to us are between 1910 and 1978. The maximum daily flow discharge within that period was 17,900 cubic feet per second (cfs) and occurred in 1917. The minimum daily flow discharge was 3620 cfs and occurred in 1977. The 50-year flood was estimated to be 19,730 cfs. The annual release from the Juniper Reservoir was given by the Instream Flow Group to be 68 cfs. From my calculations, 84 percent of the flow was diverted from the reservoir and not released downstream. From the data provided by the Instream Flow Group, it was found that the annual average release of flow was much more than the annual average inflow to the reservoir. It appears that a significant amount of flow would be diverted from Juniper Reservoir directly into the Cross Mountain Reservoir without entering the Yampa River Reach between the two reservoirs. This point should be examined rather closely. Maybe several reservoir diversion streams were used.

Change of Lily Park Reach

This reach is situated immediately downstream from the Cross Mountain Dam and thus the flow released from the reservoir will directly affect this reach. Since no detailed flow regulation for this reservoir is given, one can only discuss this problem qualitatively. For low flow, one must investigate the provision of flow depths for the fish. A great deal of vegetation growth may alter the channel shape. A botanist is needed to provide information on the relationship between flow and vegetation growth. The flood storage of the Cross Mountain Reservoir is not known, but the total storage volume appears to be rather large. The analysis of sediment samples collected at Maybell indicates that there are two types of sediment: very fine wash load and coarse material which formed the bed. In any case, it is difficult to use the cross-sectional areas of Lily Park provided by the Instream Flow Group without a field inspection and some information about its upstream and downstream conditions.

As stated previously, the major critical point of the stability of Lily Park is the stability downstream from the junction between the Little Snake River and Yampa River. Since the junction is located immediately downstream from the Lily Park section, the behavior at the junction would greatly affect the behavior of Lily Park Reach.

Table 2 shows the flow discharges and sediment loads for both the Yampa River at Maybell and Little Snake River at Lily, just upstream from its junction with the Yampa River.

Table 2. Flow discharges and sediment loads for both the Yampa River at Maybell and Little Snake River at Lily.

River	Years of Record	Drainage area (mile^2)	Annual discharge (cfs)	100 Years flood (cfs)	Suspended Load (Tons)	Bed Load (Tons)
Yampa (Maybell)	1904-1978	3910	1550	17110	0.42	0.12
Little Snake (Lily)	1923-1978	3730	575	12333	1.3	0.07

As shown in Table 2, although the drainage areas for both Yampa River and Little Snake River at the two respective stations are approximately the same, the annual flow in Yampa is much greater than that for the Little Snake River. On the other hand, the Little Snake River carries much more sediment than the Yampa River. From this, one may conclude that the Little Snake River carries a great deal of sediment to its junction with the Yampa River and the flow from the Yampa River is needed to carry these sediment deposits downstream to Green River. If one reduces the flow peaks and the annual flows through the regulation of Cross Mountain Reservoir releases, the Yampa River Reach immediately downstream from its junction with the Little Snake River may even aggrade.

All of these analyses indicate the need of a detailed analysis at the junction between Yampa River and Little Snake River. The correlation of flows between the two rivers must also be investigated. A mathematical model for this junction would be rather useful to study the various combinations of conditions.

Change of Box Elder Reaches

The effect of the two dams on Box Elder Reach probably would be minimum because the peak flows and the annual flows would be reduced and the Box Elder Reach is located downstream from a relatively steep reach.

CONCLUSIONS

1. A field inspection is needed before any detailed analysis can be made.
2. The flow regulation plans for both reservoirs should be investigated. The effects of the two dams on the two reaches may be reduced by an effective reservoir regulation plan.
3. The effect of Juniper Dam on the two reaches probably would be minor.
4. The most critical reach that should be investigated thoroughly is the Yampa River Reach immediately downstream from its junction with the Little Snake River.
5. The river bed at the Lily Park Reach may either degrade, aggrade, or remain the same depending mainly on the condition at its junction with the Little Snake River.
6. The effect of the two dams on the Box Elder Reach would probably be minimal.

RESERVOIR CONSTRUCTION AND YAMPA RIVER MORPHOLOGY

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INTRODUCTION

The Yampa River Basin lies within the southern part of Wyoming and the western part of Colorado. It was proposed that the Cross Mountain Reservoir and Juniper Reservoir be constructed on the Yampa River above Lily Park and Maybell, Colorado, respectively, as small flood retention reservoirs. The objective of this paper is to provide some preliminary assessments of the impacts of the proposed reservoirs on Yampa River morphology.

Quantitative assessment of a river's response to the construction of a reservoir can be made provided that detailed information on reservoir size and operation, flow, and sediment data collected near the reservoir, channel pattern and cross sections immediately above and downstream of the reservoir, and bed and river bank material size distribution are available. Because this information is rather limited, only qualitative assessments will be made under some assumed or hypothetical conditions. The assessments made in this paper are the author's personal opinions only and are not expressive of official policy or opinion by the Water and Power Resources Service or the United States Government.

BACKGROUND INFORMATION

The Yampa River flows through an area underlain by widespread deposits of relatively soft sedimentary rocks, bordered in part by abrupt mountain slopes, and containing isolated ridges. The most outstanding feature of this area is the meandering river cut through the mountain range to form canyons to depths of as much as 3,000 feet along the lower Yampa River. The river channel deposits along the Yampa River consist of clay, silt, sand, gravel, and boulders. Associated with the meandering river is the distinct pool and riffle sequence. The study reach is located between Lily Park and Box Elder, Colorado as shown in Figure 1. The longitudinal bed profile of the Yampa River is shown in Figure 2.

Good suspended sediment data are available at U.S. Geological Survey gaging station No. 2600 along the lower Little Snake River near Lily Park and No. 2510 along the Yampa River at Maybell between the proposed Cross Mountain Reservoir and the Juniper Reservoir. A study made by Andrews (1978) indicates that the lower Little Snake River subbasin contributes about 60% of the total Yampa River basin sediment yield, although it represents less than 35% of the area and supplies less than 3% of the streamflow. In contrast, the subbasin above Maybell, Colorado, which covers one-third of the Yampa River Basin, contributes

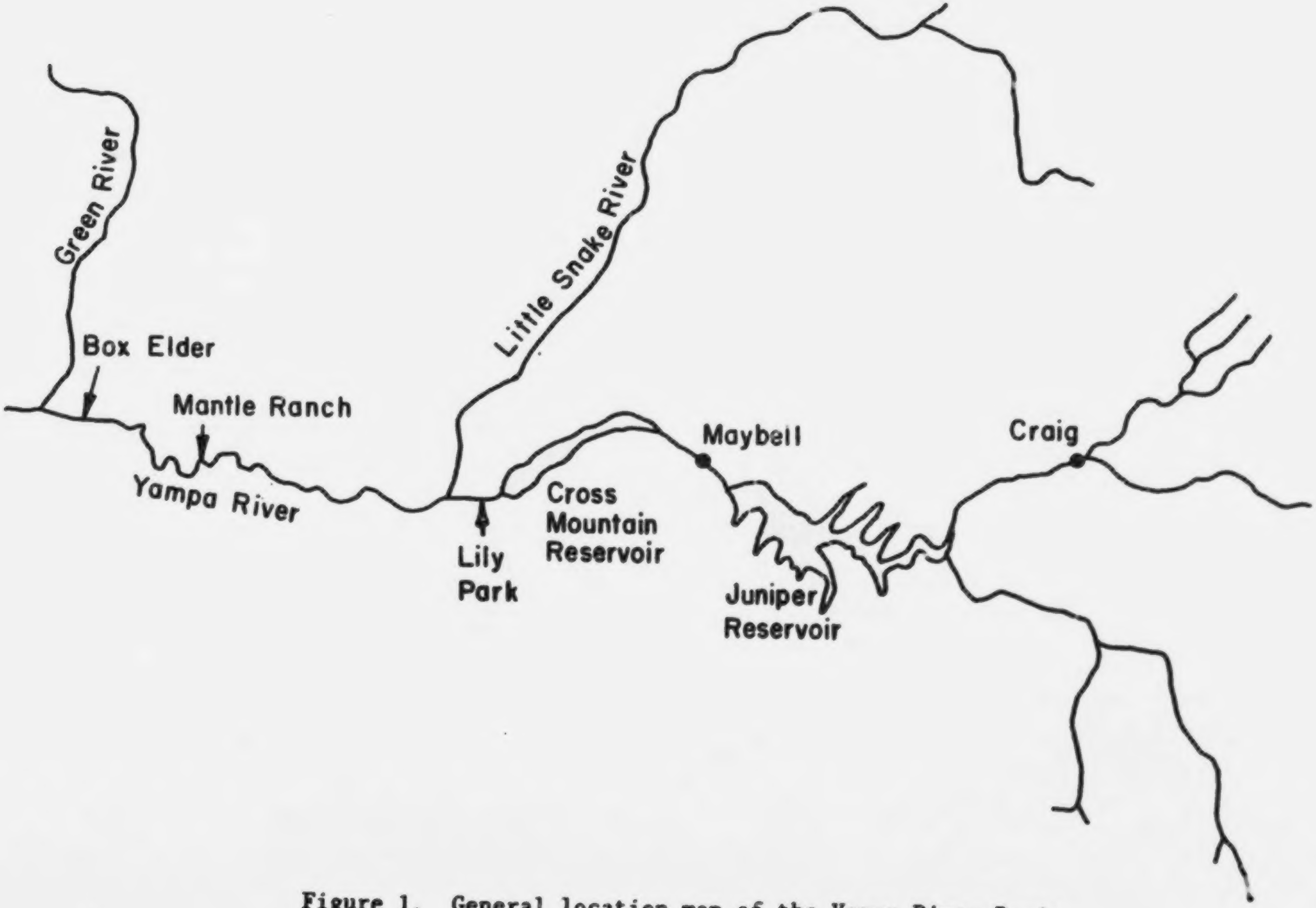

Figure 1. General location map of the Yampa River Basin.

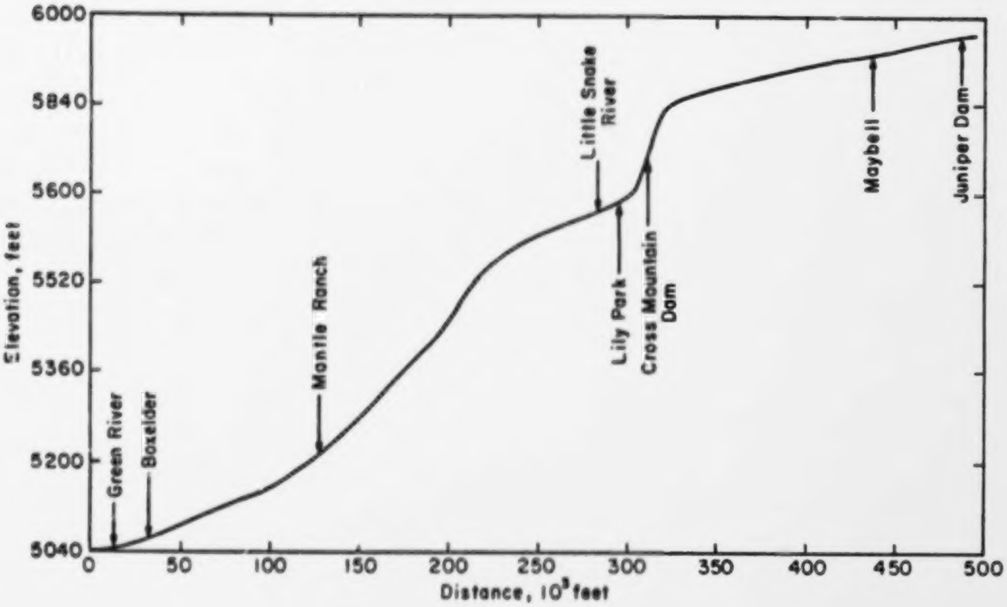

Figure 2. Longitudinal bed profile of the Yampa River.

only about 14% of the sediment yield but 76% of the streamflow. The interbed sandstones, mudstones, and shales in the study reach are relatively erodible. Based on the suspended sediment data at Maybell, the suspended loads are mainly made of materials finer than 0.062 mm. However, the bed material at Maybell has a median diameter of about 38 mm. The significant difference in size between suspended and bed material is an indication of the existence of armor layers on river bed.

The average monthly discharge from 1910-1976 and the projected average monthly discharge at Maybell with Juniper Reservoir in operation is shown in Table 1. The 10, 50, and 90% flows of the Yampa River at Maybell are about 14,000, 10,000 and 7,000 cfs, respectively.

The field survey data on channel cross sections as shown in Figure 3 and thalweg elevation of the Maybell reach indicates that the cross sections are rather symmetrical in shape and there is not a distinct pool and riffle sequence. However, the U.S. Geological Survey topographic maps indicate that the reach near Maybell is a sinuous river with sand bars. It is possible that the surveyed cross sections are not representative of the reach, and the pool-riffle sequence does exist.

The projected monthly average discharges at Cross Mountain Reservoir are shown in Table 2.

Table 1. Monthly streamflow of the Yampa River near Maybell, Colorado.

Month	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Average Flow 1910-1976 (in cfs)	21.6	20.8	18.1	16.8	18.1	41.3	156.2	380.0	323.5	80.9	23.2	14.5
Projected Average flow with Juniper Reservoir (in A.F.)	31	33	63	143	123	181	283	200	192	108	60	28

Table 2. Projected average monthly flow at Cross Mountain Reservoir, in cubic feet per second.

Month	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Discharge	452	357	266	234	202	235	356	1804	2705	2666	1091	633

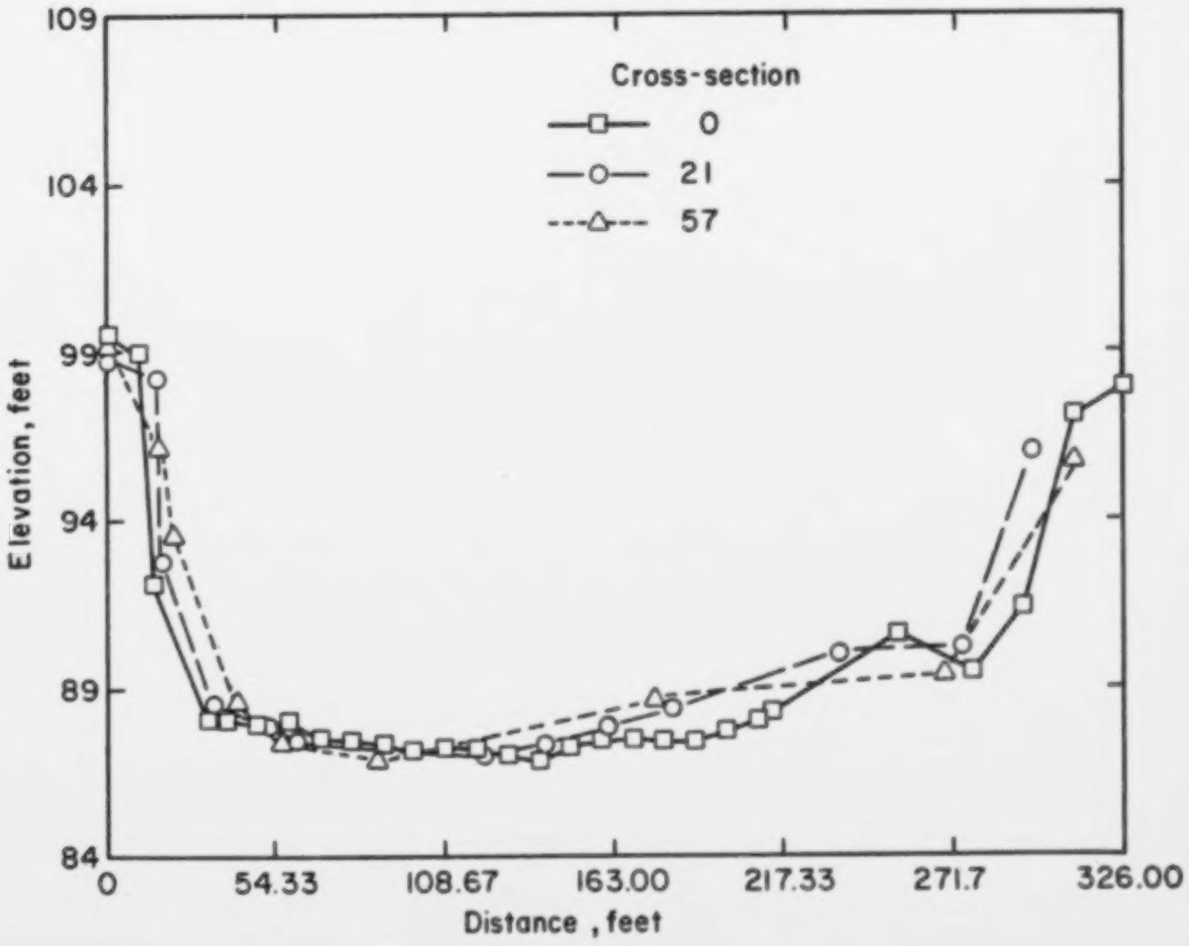

Figure 3. Channel cross sections of the Maybell Reach.

The field survey data on channel cross sections and thalweg elevation of the Lily Park reach of the Yampa River indicates that the cross-sectional shapes change from highly skewed toward one side to symmetrical and then skewed toward the other side and the cross sections have the distinct pool-riffle-pool sequence characteristics. Comparisons of thalweg profiles surveyed at discharges of 810 cfs and 4900 cfs as shown in Figure 4 indicate that there is no appreciable change of channel cross section due to changing flow conditions. This is an indication that the channel in this reach is very well stabilized by armor layer. Figure 5 shows similar results found near Mantle Ranch.

The result shown in Figure 6 is very interesting. It shows that there is no appreciable change of thalweg profile at low and median flows. However, at a high flow of 2810 cfs, the profile is higher than those measured at lower discharges. This is an indication that sediments from the Little Snake River which were deposited near the confluence of the Little Snake and the Yampa can be transported to the Box Elder reach and cause channel aggradation during high flows.

MORPHOLOGICAL CHANGES

Based on the longitudinal bed profile shown in Figure 2, morphological changes may occur in three reaches. These are the Lily Park reach, the reach near the confluence of the Little Snake and the Yampa, and the reach near Box Elder. The reach between Box Elder and the Little Snake River is very steep and the bed is either covered by armor layers or bedrock. Any sediment entering this reach will be flushed out, and no change is anticipated in the future. The only possible change of the Box Elder reach is the elimination of periodic channel aggradation during high flows since the high flows will be eliminated or reduced after the construction of the reservoirs.

The reach near the confluence of the Little Snake and the Yampa may be sensitive to the construction of the reservoirs. Future changes depend on the operation of the reservoir with respect to the hydrologic conditions of the Little Snake. Currently available data are inadequate to make a meaningful analysis and prediction.

The Lily Park reach is located immediately downstream of the Cross Mountain Reservoir and certain morphological changes are likely to occur after the completion of the reservoir. Quantitative analyses cannot be made here due to the lack of detailed hydraulic and hydrologic as well as reservoir operation data.

Qualitative predictions of morphological changes are made based on generalized background information stated in the previous section. The predicted changes emphasize the long term effects of the reservoirs and are more applicable to the Lily Park reach which is located immediately downstream of the reservoirs. These changes include meander pattern, channel configuration, substrate material, and pool and riffle sequences.

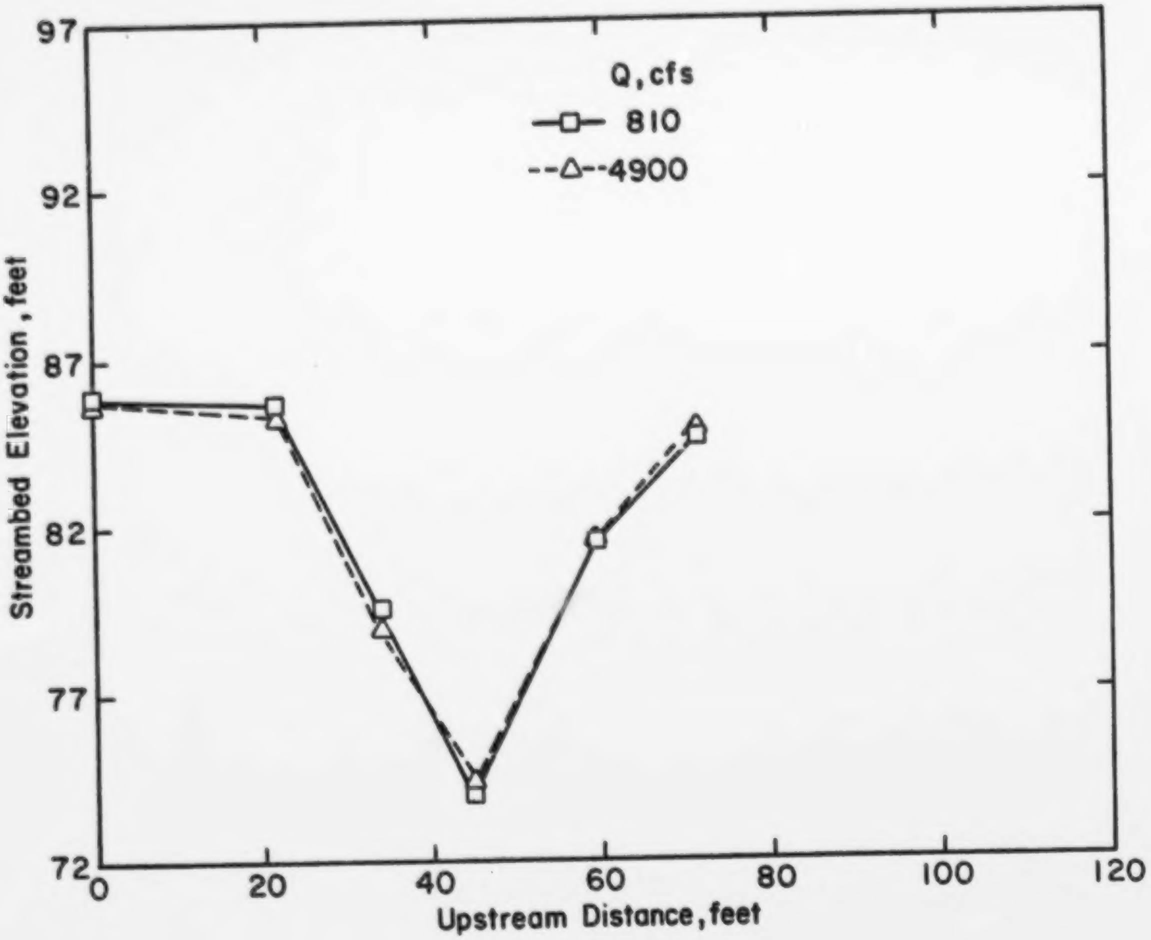

Figure 4. Variation of thalweg in the Lily Park Reach.

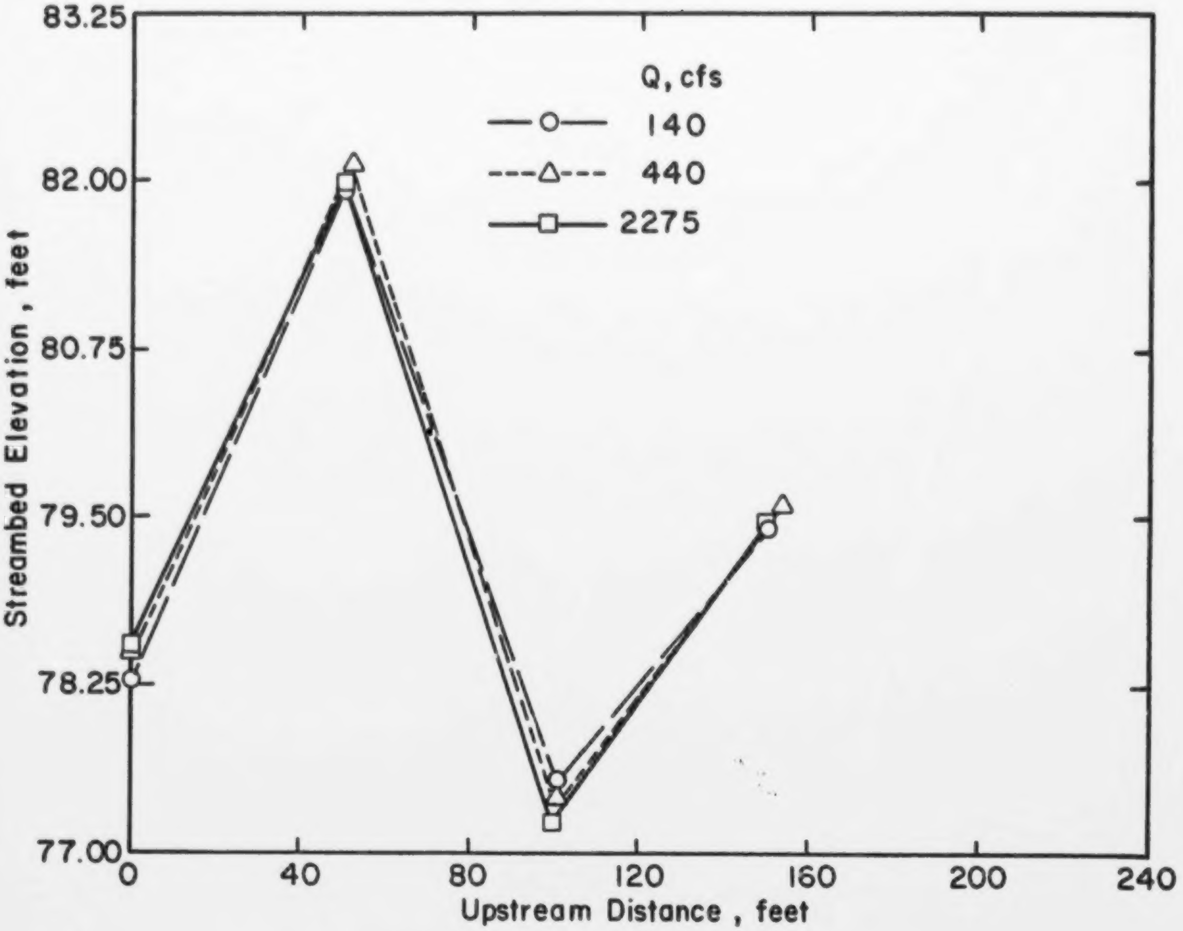

Figure 5. Variation of thalweg in the Mantel Ranch Reach.

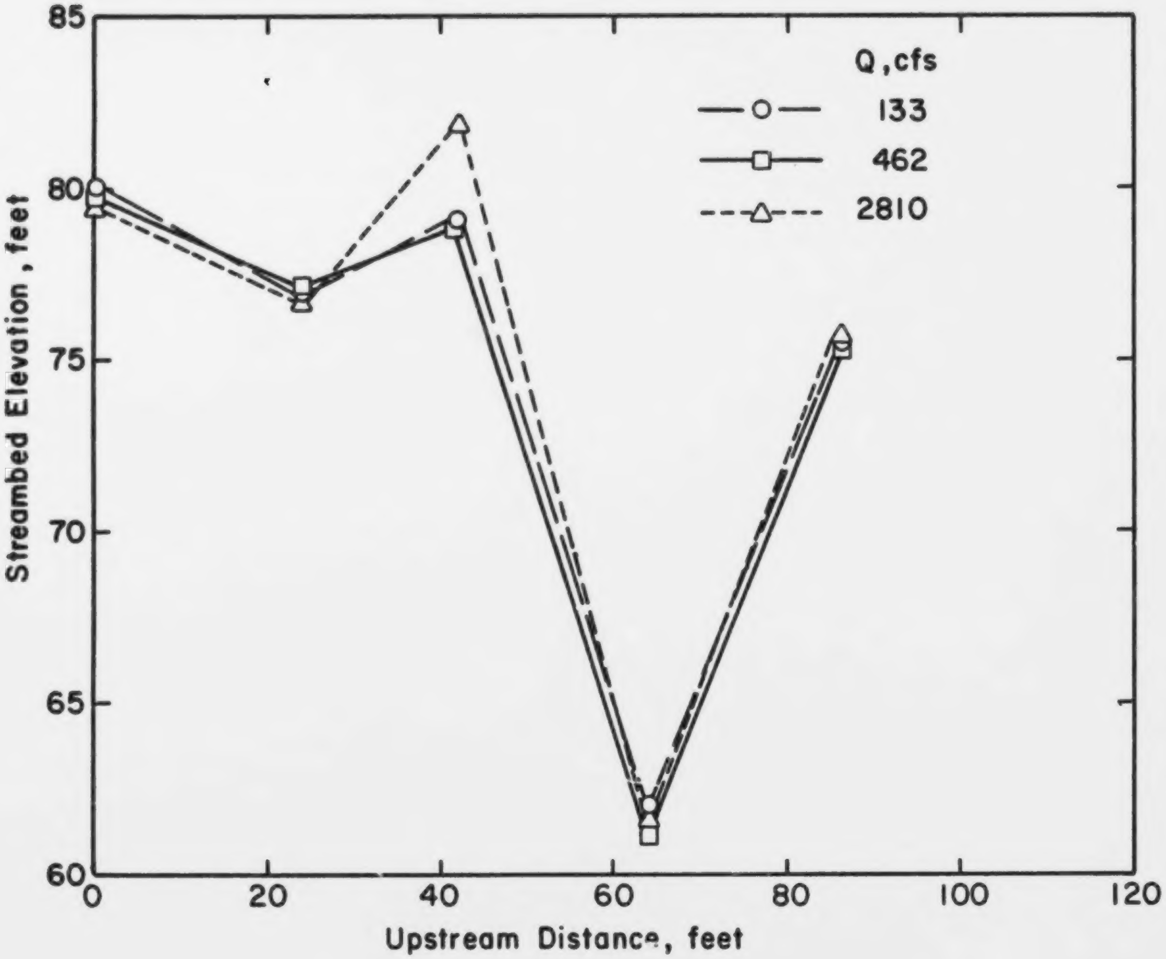

Figure 6. Variation of thalweg in the Box Elder Reach.

1. Meander Pattern. The thalweg of a river usually increases the amplitude of its meanders with decreasing water discharge. During high flow, the thalweg may cut through the point bar and reduce its amplitude of meanders. Yang (1971a, 1980) considered river meandering as a means available to a river to reduce its rate of energy dissipation. Higher discharge and slope are associated with a high rate of energy dissipation and should have smaller amplitude of meanders. After the construction of the two reservoirs, the discharge from the reservoir will be more evenly distributed over the year than the natural flow and the chance of having very high discharge from the reservoirs is very low. Because most sediment will be trapped in the reservoirs, the water released from reservoirs has the ability to erode the river bed and reduce river slope below the dams. The lack of high flows and the reduction of channel slope below a dam after construction of a reservoir should enhance the development of a more sinuous river. This is especially true for the reach immediately downstream of the proposed reservoirs because the valley width is relatively wide and bed materials are relatively erodible. However, it should be pointed out that due to the existence of coarse materials in the river bed the existing armor layer or the formation of a new armor layer should slow down the degradation process and eventually stop it. In the long run, a more symmetrical and sinuous river course may be formed.

2. Channel Configuration. Variation of channel configuration is closely related to the locations of pools at the meandering bends and crossings at the riffles. With more evenly distributed discharges from the reservoirs and a more fully developed meander pattern after the construction of reservoirs, it can be anticipated that the cross sections at the river bends would become more skewed and those at the crossing or riffles become more symmetrical.

Leopold and Maddock (1953) developed the hydraulic geometry relationships from U.S. Geological Survey gaging station records. Their relationships of channel geometry can be expressed by

$$W = aQ^b \tag{1}$$

$$D = cQ^f \tag{2}$$

in which W = channel width, D = average depth, Q = water discharge, and a, b, c, f = coefficients. Their at-a-station values for 20 gaging stations vary from 0.03 to 0.59 with a mean of 0.26, and from 0.06 to 0.63 with a mean of 0.40 for b and f, respectively. Their average downstream values are 0.5

and 0.4 for b and f, respectively. Under the assumption that a river is free to adjust its width and depth to reach an equilibrium condition, Yang et al., (1981) applied the theory of minimum rate of energy dissipation (Yang and Song 1979; Song and Yang 1980) to determine the values of b and f. The theoretical values of b and f thus determined have the same value of 0.409. This general agreement between the theoretical and observed values suggests that the theory of minimum rate of energy dissipation can be applied to predict the long term effects or the new equilibrium channel geometry under the new constraints imposed to the river after the construction of the reservoirs. Chang (1980a,b) also used the theory of minimum stream power to determine stable channel geometry and the results agreed fairly well with regime channels and natural rivers.

3. Substrate Material. As the channel degradation process continues below the proposed reservoirs, the bed materials will become coarser. Eventually, an armor layer will develop and the degradation process will stop. Because the existing bed materials abound with gravel and boulders, it should not take too long for the river bed to form an armor layer of coarser materials.

4. Pool and Riffle Sequence. River meandering and the formation of pool and riffle sequences often coexist. A river can adjust its rate of energy dissipation through meandering in the lateral direction. The formation of riffle and pool sequence was considered by Yang (1971b, 1980) as a river's self-adjustment in the vertical direction to minimize its rate of energy dissipation. This is especially true during low flows with low rates of energy dissipation and at those places where coarse materials are available to resist higher velocities at riffles. Field measurements by Stall and Yang (1972) along the Middle Fork Vermilion River near Oakwood, Illinois, during low flow showed that the rate of energy dissipation per unit weight of water flowing through two complete pool and riffle sequences was 26% less than that of an equivalent reach without any pool and riffle sequence. After the construction of reservoirs, the chance of having high discharges through the reservoirs should be significantly reduced. The low flow below reservoirs and the existence of coarse materials in the river bed should enhance the formation of more pronounced pool and riffle sequences downstream of the reservoirs. Eventually, new equilibrium pool and riffle sequences will be established.

SUMMARY AND CONCLUSIONS

A preliminary qualitative assessment of morphological changes of the Yampa River below the proposed Cross Mountain Reservoir and Juniper Reservoir was made. Because the Yampa River above the confluence of the Little Snake River has relatively low sediment load, sedimentation in the proposed reservoirs should not be a serious problem. By contrast, the sediment load in the Little Snake River is rather high. The amount of sediment which could be trapped in the reservoirs should be relatively insignificant compared with that carried by the Little Snake River. Thus, no major morphological changes of the Yampa River below the confluence of the Little Snake River should be anticipated due to the construction of the reservoirs. The predicted long term morphological changes downstream of the proposed reservoirs are summarized as follows.

1. The average amplitude of meanders should increase slightly until a new equilibrium condition is reached.

2. Along with the development of meanders, the channel cross sections will become more skewed near the bends and more symmetrical near riffles.

3. A limited amount of channel degradation below the proposed reservoirs should be anticipated until a new equilibrium profile is established. Bed materials near the bed surface will become coarser and an armor layer will be formed to stabilize the new bed profile.

4. The existing pool and riffle sequence should become more pronounced in the future.

5. Because the existing bed materials abound with gravels and boulders which can be used by the river to form an armor layer, if it has not formed already, no drastic river morphological changes downstream of the proposed reservoirs should be anticipated.

6. Because of the relatively low sediment load carried by the Yampa River above the confluence of the Little Snake River, construction of the proposed reservoirs should not have significant impact on the Yampa River morphology below the confluence of the Little Snake River.

7. The proposed reservoirs may have significant impacts on the morphological changes near the confluence of the Yampa and the Little Snake River. Available hydraulics and hydrologic data of the two rivers and the reservoir operation data are inadequate to make a meaningful prediction of future changes in this reach.

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REPORTS ON POPLAR CREEK WORKSHOP PROBLEM

POPLAR CREEK

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SUMMARY OF POPLAR CREEK DISCUSSION

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Poplar Creek is a sand and gravel bed stream located in northern California. It is a tributary to Sacramento River. Precipitation ranges from 70 inches in the headwaters to between 25 and 30 inches in the reach of interest below the proposed Dutch Gulch dam site. For purposes of the discussion the reach of concern was divided into two reaches as follows: Reach 1 between the dam and the junction of a major tributary, Dry Creek, Reach 2 below the junction of Dry Creek.

It became apparent early in the discussion that the participants were totally in agreement that the information that was provided was inadequate and that even with additional data, field work was essential to the development of an understanding of the present condition of Poplar Creek. Without this the ability to predict future changes is severely limited.

Discussion centered on the immediate response of Poplar Creek, although it was recognized that the long term adjustment would be complex with degradation followed by aggradation and vice versa, depending on the reach under consideration. Main channel scour will rejuvenate the Dry Creek tributary. The influx of sediment from this source will cause deposition below the Dry Creek confluence, and as this sediment accumulates, renewed scour is likely.

The complexity of channel response and the group's inability to predict the course of events with assurance led to the suggestion that experimental and field studies be carried out on the effect of dam construction on a range of channel types. It was also suggested that available information on this topic be assembled in a volume of case studies.

The possible effects of bedrock controls and of variations of the erosional resistance of the valley alluvium was considered to be of sufficient importance that there should be borings to obtain representative samples of the alluvium.

The evaluation of the reach by each expert relied heavily on his past experience, and at first a qualitative approach was followed, although standard sediment-transport and hydraulic equations were used to support conclusions.

The considerable experience of the participants resulted in a healthy degree of caution concerning channel response. All recognized that each river is sufficiently different from others that generalizations about channel response are hazardous. Only after field inspection and collection and analysis of additional sedimentologic, hydraulic, geologic, hydrologic and geomorphic data can predictions of channel response be made with any degree of confidence. However, even with these data, the river reach must be considered as a component of a larger fluvial system. Historical studies of channel behavior through time also should be made if old maps and photographs can be located.

Because of the limited information available each participant used several approaches to evaluate the channel and its likely response. For this reason the approaches were relatively straight forward and simple in concept. For example, sediment transport equations were used to estimate depths of scour and the potential for armor development. Hydraulic geometry equations were used to estimate changes of channel dimensions. Histories of gravel-bed channel response elsewhere were used to suggest the changes of channel morphology and sediment character that could be anticipated.

All participants relied heavily on their past experience, and each stressed the need for careful map, aerial photograph, and field studies. Because of the complexity of the long term response of the channel, the use of existing equations alone is inadequate, because they only provide information on the short term response of the channel. For example, tributary response to main channel adjustment is critical. Influx of sediment from both Dry and Little Dry Creeks will greatly complicate the response of Poplar Creek. Future changes of Sacramento River position, laterally or vertically, can also significantly affect Poplar Creek in unanticipated ways.

A further complication is the operating schedule of the dam. Large water releases could exacerbate the downstream problems, but greatly reduced flows could permit the tributaries and the Sacramento River to become dominant factors in determining reach characteristics.

In spite of the desire for further information, the group was in general agreement that at least initially Reach 1 would degrade and armor, whereas Reach 2 would aggrade, as a result of initial degradation and rejuvenation of Dry Creek. During the discussion the reporter attempted to summarize the conclusions of each participant concerning changes of channel morphology (width, depth, shape, sinuosity, and pool and riffle spacing) channel behavior (meander shift, cutoffs, bed elevation change) as well as the potential for vegetation encroachment into the channel and changes of sediment characteristics (size and potential for armoring). The majority of the participants agreed that channel width, depth, and width-depth ratio would decrease. Sinuosity would probably increase slightly. Opinion was divided concerning changes of pool and riffle spacing, meander shift, and cutoffs. Streambed elevation will generally decrease in Reach 1 and increase in Reach 2. Vegetation is expected to encroach on the channel, and bed material will increase in size and an armor will develop in Reach 1. Bed material size should decrease in Reach 2 as aggradation occurs. The evaluations of the participants are summarized in Table 1.

Table 1. Poplar Creek participants evaluations

Channel Change	Andrews	Emmett	Hey	Parker	Vanoni	Winkley
Depth	-15%	- slight	cyclic change + and -	R1 -	?	cyclic change + and -
Width	-20%	0	R1 - R2 +	-	-	cyclic but mainly -
Shape (w/d)	-	R1 - R2 +	?	- slight	?	?
Sinuosity	+	0	+	0	+	+
Pool and riffle spacing	+	0	R1 - R2 +	0		
Meander shift	+	0	+	0	+	+
Cutoffs	+	0	?	0		+
Bed elevation	R1 - R2 0	R1 - R2 +	R1 - R2 [illegible]	R1 - R2 + LT -	+	cyclic + and -
Vegetation encroachment	+	?	+	+	+	+
Bed material size	+20%	R1 + R2 - no	R1 + R2 -	R1 + R2 0	+	cyclic + and -
Armor	+	0	R1 + R2 -	R1 + R2 0	+	0

Explanation
R1 = reach 1
R2 = reach 2
LT = long term response
+ = increase
- = decrease

ASSESSMENT OF STREAM CHANNEL RESPONSE TO ALTERED STREAMFLOW AND SEDIMENT LOAD

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INTRODUCTION

The assessment of stream channel response to altered streamflow and sediment load depends upon both an accurate description of the natural or initial conditions of the channel as well as the analytical methodology employed. It is quite impossible by any method to predict the hydraulic characteristics of a stream channel following changes in streamflow and sediment load without some knowledge of natural stream conditions. The accuracy and specificity of the predicted channel response improves as the hydraulic characteristics of the unaltered stream are better known.

With this in mind, the available information describing the hydraulic characteritics of Poplar Creek is wholly inadequate to make reasonable predictions of the channel response following the construction of the Dutch Gulch Dam. This deficiency limits the assessment of probable stream channel response far more than our understanding of fluvial processes, or our ability to apply this knowledge.

It has been my personal experience when considering stream channel response that in a majority of situations the hydrologic description of the affected stream is insufficient to address the significant issues of land use change. The Poplar Creek example is, in fact, quite typical of the situation one faces in evaluating stream channel response to disturbance. Although frequently there is considerable information describing the hydrology of the stream, i.e. annual runoff, daily discharges, annual peak flows, etc., this information is not sufficient to describe the channel morpholoy.

In many instances, an adequate hydraulic and geomorphic description can be obtained rather easily and without great expense. The essential elements of such a geomorphic description are:

1. A study reach of a length of at least 30 channel widths, so that three to four pool and riffle sequences and meanders are described.

2. Several cross sections (10-15) that describe the variation in channel morphology. Each cross section profile should show the bankfull elevation, vegetation, and size distribution of the bed material.

3. Surveyed longitudinal profiles of streambed, water surface, and bankfull elevations. Longitudinal profiles determined from maps are generally inadequate. In constructing these profiles, channel features should be surveyed at points no more than every one-half channel width apart.

4. A sketch map of the study reach showing the location of surveyed cross sections and longitudinal profiles, and accurately portraying the channel pattern and alignment.

5. Photographs of the study reach are quite valuable, especially sequential aerial photographs. When assessing the future hydraulic adjustment of a river, it is extremely helpful to know something about the character of the river over the past 20-40 years. Aerial photographs are the most readily available and useful sources of the information.

All of this information could have been collected for the Poplar Creek study reach by a two person surveying party in a week. The total cost would be quite modest; approximately $1,000, which is miniscule compared to that of the project. Without this information, the analysis of stream channel response is hampered much more by insufficient knowledge of the stream than the uncertainty of the theoretical or analytical methodologies.

HYDRAULIC CHARACTERISTICS OF POPLAR CREEK STUDY REACH

Due to the lack of hydraulic and geomorphic information about the Poplar Creek study reach, it is necessary to make several assumptions regarding the character of the channel. Each of these assumptions will be noted. To the degree that these assumptions do not agree with the actual character of the Poplar, the anticipated response will be in error.

Several surveyed cross sections of the study reach were provided. It appears that these sections were surveyed for purposes of a flood routing study, because critical geomorphic features were not identified. In each case, an "active" channel was noted; however, it is unclear whether the "active" channel is, in fact, the self-formed or bankfull channel. For lack of a better estimate of the bankfull channel width, however, I have assumed that the "active" channel width is approximately the bankfull channel width. The average width of these cross sections is 460 feet.

Several sediment size analyses were provided. Two samples appear to have been collected at some of the cross sections surveyed by the U.S. Corps of Engineers. These are identified as edge of stream and overbank samples. In addition to the sediment size analysis performed by the Corps, a few bed-material size analyses at a gaging station operated by the U.S. Geological Survey were also provided. The bed-material samples analyzed by the U.S. Geological Survey indicated a median diameter of approximately 4 mm, whereas the Corps of Engineers' samples indicate a median of 28 mm. These values appear to be in conflict, unless the U.S. Geological Survey samples were taken from a pool, and the Corps samples were taken from a riffle or bar. My personal experience with other streams draining the west side of the Sacramento Valley suggests that this is the likely explanation for the discrepancy. The average bed-material size distributions assumed for riffles and pools are (Table 1):

Table 1. Bed-material size distributions for riffles and pools.

	Bed Material Size Riffles (mm)	 Pools (mm)
D_{16}	3	.6
D_{35}	15	1.8
D_{50}	28	4.0
D_{65}	43	8.5
D_{84}	67	20.0
D_{90}	85	24.0

A generalized stage-discharge relation for the study reach was computed using a modified form of the Keulagen's logarithmic velocity relation proposed by Bray (1979):

$$\frac{\bar{u}}{\sqrt{gds}} = 3.56 + 6.11 \log \frac{d}{D_{90}}$$

An average slope of 0.0018 was determined from topographic maps for the study reach.

A bedload sediment transport vs. discharge relation was computed using the derived stage-discharge relation, the assumed pool bed-material size distribution, and the Meyer-Peter, Müller bedload transport equation. A few measured bedload transport rates and sizes determined by the U.S. Geological Survey using a Helley-Smith bedload sampler are in good agreement with the computed relation between bedload transport and discharge.

Synthesized daily mean discharge flow-duration relations at the Dutch Gulch Dam site were provided for the natural and regulated conditions. The computed bedload transport vs. discharge relation was combined with these flow-durations to determine the effective discharge for the natural and regulated conditions. Andrews (1980) showed that the increment of discharge that transported the largest fraction of the mean annual sediment load over a period of years, i.e. the effective discharge, was approximately the bankfull discharge; thus the effective discharge measurements will be assumed to be reasonable estimates of the bankfull discharge. The computed unregulated effective discharge was 10,000 cfs and the computed regulated effective discharge was 7,200 cfs.

In both the regulated and unregulated conditions, the bankfull discharge will be equalled or exceeded approximately 0.3% of the time, or slightly more than one day per year. This is within the range one would expect for such a drainage basin.

The unregulated mean bankfull hydraulic characteristics of the study reach were determined from the estimated bankfull width, computed stage-discharge relation, and computed bankfull discharge. Estimated bankfull hydraulic characteristics are: velocity, 5.2 cfs; mean depth, 4.2 ft; and width, 460 ft.

The regulated mean hydraulic characteristics were determined using the unregulated bankfull hydraulic characteristics and the hydraulic geometry equations developed by Leopold and Maddock (1953).

$$\bar{u} = 2.07\ Q^{0.1}$$

$$\bar{d} = 0.106\ Q^{0.4}$$

$$w = 4.6\ Q^{0.5}$$

The regulated bankfull hydraulic characteristics of the study reach were determined by solving these hydraulic geometry relations, assuming a bankfull discharge of 7,200 cfs. Thus, the computed bankfull hydraulic characteristics as a result of reduced streamflow due to the construction of Dutch Gulch Dam are: mean velocity, 5.03 cfs; mean depth, 3.68 ft; and width, 390 ft.

This analysis has neglected some significant factors affecting the hydraulic geometry of Poplar Creek. Regulation of Poplar Creek by Dutch Gulch Dam will reduce the duration of very large discharges and increase the duration of intermediate discharges. Both of these changes will tend to stimulate the encroachment of vegetation into the channel. As a result, the channel will probably be somewhat narrower and deeper than indicated by the hydraulic geometry relations.

SEDIMENT TRANSPORT AND STREAMBED DEGRADATION

As described above, the transport rate of bedload through the study reach was computed for several discharges from the derived stage-discharge relation, the assumed pool bed material size distribution, $D_m = 7$ mm and $D_{90} = 40$ mm, and the Meyer-Peter, Muller bedload transport equation.

The mean annual bedload discharge of Poplar Creek near Olinda was computed to be 60,000 tons under the natural conditions. Assuming that the size distribution of bed material is the same, and using the projected flow duration for the Poplar Creek below the Dutch Gulch Dam site, the mean annual bedload discharge will be reduced to approximately 26,000 tons/year. Construction of the dam will trap the entire 60,000 tons/year of bedload material that is presently transported past the Olinda gage. Thus, the quantity of sediment transported out of the study reach will exceed the supply and the streambed will degrade as long as the supply of sediment from downstream tributaries is less than 26,000 tons/year.

The mean annual bedload-discharge per unit drainage area upstream of the Olinda gage is 150 tons/mi^2-year. Estimated mean annual bedload discharge per unit drainage area for other gaging stations along the west side of the Sacramento Valley ranges from 10 to 200 tons/mi^2-year. The drainage area contributing to the study reach is 80 mi^2. Therefore, the quantity of bedload material supplied to the study reach by tributaries will be approximately 12,000 tons/year. (In the following discussion, it will be assumed that the particle size distribution of the sediment supplied to the study reach by tributaries is the same as the bed-material found in the pools at low flow under present conditions.) Thus, throughout the study reach, the expected annual bedload transport rate will exceed the supply of sediment from tributaries. Immediately below the dam and above the first tributaries, the expected excess of transport over supply will be 26,000 tons/year. This deficiency will decrease downstream to approximately 14,000 tons/year at the downstream end as tributaries supply sediment to the study reach. As a result, relatively small, transportable sediment particles will be eroded from the streambed, and the channel will degrade. Concomitantly, the bed-material will become coarser, and the bedload transport rate will decrease. Ultimately, the quantity of sediment transported will equal the quantity of sediment supplied to the channel (about 12,000 tons/year) at which time the streambed degradation will stop.

The average depth of streambed degradation can be estimated by calculating the maximum size of particles that will be transported by the reduced streamflows following construction of the dam. One method for calculating the threshold particle diameter is to set the Meyer-Peter, Muller bedload equation equal to zero (i.e. no transport), (U.S. Bureau of Reclamation 1973). The calculated threshold particle diameter in Poplar Creek at a bankfull discharge of 7,200 cfs is 27 mm.

As noted previously, the Poplar Creek study reach appears to have a fairly well developed pool and riffle sequence at this time. The riffles are composed of relatively coarse material (D_{50} = 28 mm), whereas the pools have relatively fine bed material (D_{50} = 4 mm). Because the median diameter of riffle bed material is already as coarse as the threshold particle size that will be transported by the regulated streamflow, degradation of these riffles will be slight, about .25 feet. Nearly all of the pool bed material, however, is smaller than the threshold particle size, and consequently, significant degradation of the streambed through pools, perhaps three feet or more, is likely. Thus, over the next few decades, it seems probable that there will be an enhancement of the pool and riffle sequence in the study reach. However, because the regulated streamflows will be insufficient to transport the size of material that presently makes up the riffles, the existing riffles will become relics from the unregulated hydraulic conditions. Natural lateral migration of the stream channel will, in time, develop an entirely new channel. It appears likely that the new channel will also have a pool and riffle sequence, but the riffle bed-material will be smaller ($d_{max} \approx 27$ mm), due to the reduced magnitude of streamflows.

In spite of the general trend towards degradation, there may be localized areas of aggradation at the mouth of major tributaries, particularly Little Dry Creek and Dry Creek. At present, there is a significant bulge in the longitudinal profile of Poplar Ceek where Little Dry Creek and Dry Creek join. The impact of these tributaries on Poplar Creek will depend on how severely flood flows in Poplar Creek are reduced by the Dutch Gulch Dam, the relative timing of peak flows in Poplar Creek and the tributaries, and the quantity of sediment supplied by the tributaries that is larger than can be transported by Poplar Creek. The tributaries may well have an extremely significant impact on the study reach following construction of the Dutch Gulch Dam. Consequently, it is essential to define the hydrologic and sedimentologic characteristics of the tributaries.

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POPLAR CREEK, CALIFORNIA: A BRIEF EVALUATION OF RIVER BEHAVIOR

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INTRODUCTION

Poplar River is a sandy, gravel-bed river in northern California. Proposed river-management schemes would alter the duration of natural streamflows by reducing peak flows and releasing impounded waters at near and below bankfull stages. Channel response to the redistribution of streamflow is related primarily to bed-material transport.

The present report attempts to predict some of the channel responses to the proposed water-management schemes. The analysis is based on existing data; no field visit to the site was made. Existing data consist of:

1. Selected topographic maps (USGS),
2. Synthesized high flow frequency curves (USCE),
3. Synthesized natural and project flow duration curves by station correlation (unknown),
4. Channel cross-sectional surveys (USCE),
5. Selected streamflow, water quality, and sediment data (USGS 1979),
6. Summary of discharge measurements (USGS), and
7. Flow-duration curves (USGS).

Items 1-4 were provided by the U.S. Fish and Wildlife Service, item 5 is a published library reference, item 6 was solicited from the U.S. Geological Survey office in Sacramento, and item 7 was retrieved from the WATSTORE computer files of the U.S. Geological Survey.

Few of the supplied data were useful to make a river analysis. An experienced river engineer, spending one day in the field, could have collected substantial data to support a more rigorous analysis.

With inadequate data availability, the approach taken in the analysis is an application of approximating channel-geometry relations and comparison with case history data. An assumption is that the behavior of Poplar Creek is not too unlike that of similar rivers. Results of the analysis are adequate for planning purposes, but should not be used as a basis for engineering design.

DESCRIPTION OF STUDY REACH

The study reach is some 50,000 feet in length, extending from Dutch Gulch to the confluence with a principal tributary fork. Drainage area at the upstream end of the reach is some 400 square miles. Bed material is primarily coarse sand and gravel, and is characterized by d_{35} = 1.0 mm, d_{50} = 1.8 mm, d_{65} = 2.2 mm, d_{84} = 8 mm, and d_{max} = 32 mm (USGS 1979:107).

Mean annual precipitation is about 40 inches per year; approximately one-third runs off giving a mean annual discharge of 432 cfs (USGS 1979:102) at the upstream end of the reach. The stream hydrograph is flashy with a peak flow nearly 100 times the mean annual flow (data item 2). Whereas many rivers have peak flow only slightly greater than 1-day flow and only about 1.5 times 30-day (Emmett 1975), Poplar River has peak flow about 1.5 times 1-day flow and about 7 times 30-day flow (data item 2).

Bankfull discharge can be approximated as the 1.5-year flow (Emmett 1972, 1975, Leopold et al. 1964), or about 5,000 to 6,000 cfs (data item 2). The average width of mobile channel is about 175 feet (data item 4, data item 6, and USGS 1979:107, using width data given and applying techniques of Emmett (1975). Bankfull depth and velocity, approximated by channel-geometry techniques (Emmett 1972, 1975), are about 4.5 feet and 7.0 fps, respectively. These numbers multiply to give 5,500 cfs as the bankfull discharge. Although regulation will reduce peak flows by a 3- to 4-fold factor (data item 3), bankfull discharge will be changed in flow duration by only 0.2%, from about 0.8% of the time to 0.6% of the time (data item 3). In reality, only peak flows at the upper 0.1% of time are being eliminated by regulation. Further, comparison of the actual flow duration for the period of operation, 1972-1979 (data item 7), and the synthesized project flow duration (data item 3) indicates no significant differences in the flow duration between project operation and the actual flows in eight years of data collection (Table 1).

The river drops 100 feet in the 50,000-foot study reach (data item 1), giving a nearly uniform slope of 0.002 ft/ft. Based on a bankfull discharge of 5,500 cfs, this value of slope would indicate a mildly to moderately braided channel, reasonably straight in course but unstable in smaller-scale pattern (Leopold 1964). That is, low-water channel configurations are different from one season to another. Size similarity of bedload and bed material (USGS 1979:107) indicates the river is competent to transport most sizes of sediment occurring on the streambed.

Table 1. Discharge summary.

Percentage of time	Discharge (cfs x 10^3)		
	Actual Record	Synthesized Natural	Synthesized Project
0.03	10.9	15.8	10.9
0.05	10.3	13.4	10.3
0.08	9.6	11.8	9.8
0.1	9.3	11.1	9.6
0.2	8.0	9.2	8.8
0.4	6.5	7.4	7.0
0.8	5.3	5.4	4.7
1.6	3.4	3.7	2.8
3.2	2.2	2.4	1.6
6.4	1.5	1.4	1.0
10.0	1.0	1.0	.8

BED-MATERIAL TRANSPORT

Bedload generally relates to a stream-power term or for constant slope and little changes in width, to the stream discharge. More precisely, the relation is proportional to about the three-halves power of the discharge in excess of the discharge to initiate movement of bed material. Using case history data for similar conditions (Emmett 1976; Emmett 1978; Leopold and Emmett 1976) and with some validation of the case history data (Emmett 1980; Emmett and Thomas 1978), at bankfull discharge, bedload transport in Poplar Creek is about 2000 tons/day. At twice bankfull stage, the approximate maximum regulated flow, bedload transport is about 5,000 tons/day. At half bankfull, bedload transport is about 600 tons/day.

Approximately 90% of the bedload transport occurs in 10% of the time. Applying the bedload function to the upper 10% of the synthesized flow-duration values (data item 3), the unregulated flow transports about 23,000 tons of bedload and the regulated flow transports about 18,000 tons. Using the actual flow duration (data item 7), the bedload transported in 10% of the time is about 20,000 tons. A total annual average of bedload transport for the unregulated stream is estimated at 25,000 to 30,000 tons. This compares to a value of 40,000 tons during water year 1978 (USGS 1979, p. 107) when streamflow was two-thirds greater than normal.

CONCLUSIONS

Assuming a reasonable trap efficiency for the proposed impoundment, the stream would desire to transport approximately 20,000 tons of bedload which is not available. Degradation below the reservoir might occur with a concurrent coarsening of the bed by depletion of smaller particles. This will lead to armoring, and any degradation will be modest. Downstream, the stream might not be able to transport all material supplied by its tributaries and modest aggradation might occur in these reaches. Little or no change in other channel characteristics would appear other than a decrease in near-annual floodplain inundation. Adherance to the operating rules (data item 3) imposed on this analysis should minimize channel changes. Monitoring of downstream channel changes could help in any revision needed in operating policy to help eliminate adverse effects.

It is emphasized that this analysis is preliminary and should be used only to evaluate some techniques available for assessment of river behavior.

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CHANNEL ADJUSTMENT TO RIVER REGULATION SCHEMES

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INTRODUCTION

River regulation, including interbasin water transfers, has been advocated as a means of equalizing water supplies and demands in England and Wales. In this way the river is used as a natural aqueduct and it obviates the need for the construction of costly tunnels and pipelines linking the remote source with the demand center.

Rivers can be regulated either by releases from upland impounding or pumped storage reservoirs, interbasin transfers, or by the discharge of pumped ground water. During the last decade there have been several developments of this type in England and Wales and further schemes are either under construction as, for example, the Kielder Reservoir on the River North Tyne or, like the Craig Goch Reservoir on the River Wye, at the planning stage.

By modifying the flow regime and sediment transport characteristics of a river, such developments may be responsible for erosional and depositional activity as the channel adjusts to the new flow conditions. In order to assess whether regulation will cause unnatural amounts of erosion or deposition and, if so, predict the response of the channel to river regulation, it is necessary to briefly consider the mechanisms controlling the development of natural channels.

CHANNEL ADJUSTMENT MECHANISMS

Process-Response Models

Channels respond to variations in discharge and sediment load through the operation of governing or process equations. These equations define the mechanisms whereby channels adjust their hydraulic geometry through erosion and deposition. For gravel-bed rivers there are seven unknowns, velocity, hydraulic radius, wetted perimeter, maximum flow depth, slope, sinuosity and meander arc length, and it follows that there must be seven governing equations. These are the continuity, flow resistance, sediment transport, bank competence, and sinuosity equations. Two further equations are required for sand-bed channels due to the development of bed forms. Provided these physical processes can be specified mathematically, then their simultaneous solution will define the dimensions of alluvial channels given the discharge, input sediment load, bed and bank material, and the valley slope (Hey 1978).

The operation of channel feedback mechanisms enables a section of channel to interact with neighboring reaches to adjust its input conditions. This occurs because the processes of aggradation and degradation, through their action on channel slope, flow depth, and bed sediment size promote further instability. The stabilizing influence of negative feedback mechanisms results from the effect of upslope erosion or deposition on the input sediment load. During erosion, drawdown effects are responsible for the upstream migration of erosional activity and the increased caliber and volume of the input load gradually reduce the erosion rate and eventually re-establishes stability. This will be only a temporary phenomenon because the increase in sediment load, as erosion progresses further upstream, will immediately initiate deposition. Similarly the depositional phase will tend to stabilize as the reduced caliber and volume of the input load, resulting from backwater effects promoting aggradation upstream, can be transmitted through the cross section.

Once again the stable condition will be only a temporary phenomenon since the continued reduction in input load, due to the upstream progression of the aggradational phase, will precipitate further erosional activity. The river will then rework some of the sediment which had temporarily been stored in the channel. This indicates that there will be a constant interplay between erosional and depositional activity in time and space. The magnitude and frequency of the oscillation will, however, decline and a regime type condition will eventually become established (Hey 1979).

Dominant Discharge

Although rivers adjust their bankfull shape and dimensions to a range of flows it has been argued that the steady discharge which produces the same gross shapes and dimensions as the natural sequence of events is the dominant discharge (Nixon 1959). Flume experiments and observations of natural channels indicate that flows at, or about, the bankfull stage are the effective ones for channel forming processes (Ackers and Charlton 1970).

Wolman and Miller (1960) in their study of the magnitude and frequency of sediment transport processes showed that the frequency of occurrence of the flow transporting most sediment and the bankfull discharge were approximately equal and this explains why bankfull flow appears to be the dominant or channel forming discharge. The magnitude and frequency of the flow transporting most sediment is dependent on the flow regime and the nature of the sediment transport processes (Figure 1). For stable gravel-bed rivers this is the 1.5 year flood (annual series) while for sand-bed channels the return period of bankfull flow is lower. Thus, although extreme discharges are individually responsible for transporting large volumes of sediment, it is the smaller more frequent flows which collectively transport the largest volume of sediment over a period of years.

Any change in the input sediment load or the frequency of flows which can transport sediment will cause erosion or deposition. During periods of instability the channel dimensions and the bankfull discharge adjust to the flow doing most erosion or deposition. When eroding the capacity of the channel and the return period of bankfull flow are increased and vice versa when depositing. The flow variability is also influenced by these systematic changes in channel capacity. Depositional activity reduces channel capacity, increases flood attenuation, and reduces the flow variability; while erosion increases flow variability (Hey 1975a).

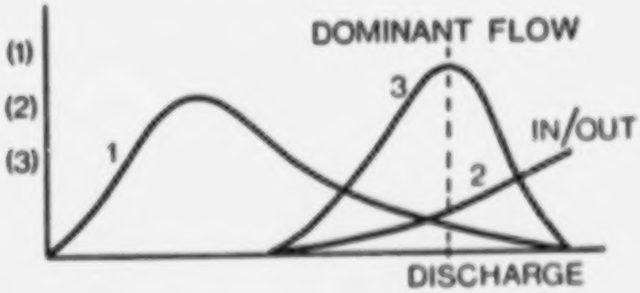

Figure 1. Flow Transporting Most Sediment
Key (1) Frequency, (2) Instantaneous Sediment Discharge,
(3) Total Sediment Discharge.

EFFECT OF RIVER REGULATION ON CHANNEL STABILITY

River regulation and interbasin transfers, through their influence on flow variability, mean flows, and sediment transport rates can have considerable effect on the magnitude and frequency of sediment transport processes. Consequently the channel downstream from a regulation point could be susceptible to either erosion or deposition as it adjusts its capacity and hydraulic geometry to the new flow regime (Hey, 1975b).

With surface water reservoirs, regulation can be achieved either directly below the dam site, or at a point farther downstream through the operation of a river intake/outfall. In both cases flow variability will be reduced downstream from the regulation point, because the magnitude and frequency of the flood flows are reduced in order to decrease the number of exceedingly low flows, and the sediment transport characteristics of the channel will be considerably altered. Mean flows will also be modified if interbasin transfers are involved.

Reservoirs

There are many examples of scour effects downstream from dam sites. Principally it results from a reduction in sediment load consequent upon the construction of the reservoir (Komura and Simons, 1967), although modification to the flow regime can be a considerable influence (Gregory and Park 1974).

Immediately downstream from the dam site the channel will initially retain its ability to transport sediment at certain flow stages. As input load is zero erosion will commence which, in turn, will reduce the slope of the channel and the bed shear stress and increase the average size and variability of the bed sediment. Consequently the ability of the channel to transport sediment will progressively decline until all flows are below the threshold discharge for bed material movement (Figure 2). Material flushed out by this process will be deposited lower downstream. Eventually the progressive reduction in sediment output below the dam will cause erosion at sections further downstream and the process is repeated. Even if the channel immediately below the dam site is cut in bedrock, erosion will be experienced in the alluvial reaches of channel further downstream if regulated flows can transmit sediment. If tributaries are unregulated, sediment entering the main channel may not be transmitted by the reduced flow levels and this can result in localized deposition. Reduced main river levels can also promote erosion on the tributary and this can exacerbate the depositional activity on the main stream.

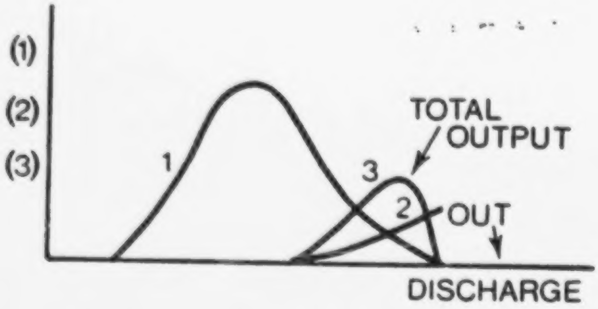

Figure 2. Effect of Regulation on Total Sediment Discharge Below Dam Key (1) Frequency, (2) Instantaneous Sediment Discharge, (3) Total Sediment Discharge.

Natural channel stability can be maintained provided releases are made below the minimum threshold discharge for bed material transport or bank erosion downstream from the reservoir. The maximum release at the dam will therefore depend on the downstream increase in natural flows when releases are being made.

River Intakes/Outfalls

Pumped storage reservoirs are increasingly being used to regulate flows in alluvial channels through the operation of river intakes/outfalls. Interbasin transfers are often associated with such schemes. In this type of development both the flow regime and sediment transport characteristics of the channel are changed dramatically over a short stretch of channel and, unlike the reservoir situation, the sediment input is not necessarily zero.

If regulation alters the number of flows that can transport sediment, channel instability will result. This will be most severe when erosion or deposition occurs at the intake/outfall point because it precipitates instability both upstream and downstream. However, provided channel feedback mechanisms operate very slowly, dynamic stability may be achieved at the intake/outfall site when aggradation balances degradation over a number of years (Figure 3). Downstream the dominant discharge would be defined by the flow transporting most sediment (output), while upstream it would be defined by the flow transporting most sediment (input). If feedback mechanisms operate very rapidly during periods of instability and promote bank erosion and a change in the plan geometry of the channel, this would permanently alter the hydraulic characteristics of the channel, and preclude the re-establishment of the original condition. Equilibrium would be achieved in these circumstances only after the channel had modified its hydraulic geometry and reduced its sediment transport capacity so that all releases were below the threshold discharge for sediment transport.

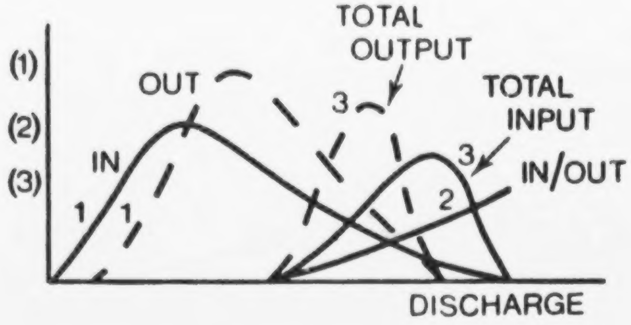

Figure 3. Effect of Regulation on Total Sediment Discharge at a River Intake/Outfall
Key (1) Frequency, (2) Instantaneous Sediment Discharge, (3) Total Sediment Discharge

Provided regulation does not alter the frequency of flows above the threshold discharge for bed material transport and bank erosion downstream from the intake/outfall, the natural character of the channel can be preserved. The associated maximum regulated flow at the intake/outfall will depend on the downstream rate of increase in base flow when abstractions and releases are being made. If natural flows do not increase significantly downstream when regulation is taking place, the maximum regulated flow at the outfall is defined by the minimum threshold discharge for bed material transport or bank erosion downstream from the intake/outfall.

PREDICTION OF CHANNEL RESPONSE TO RIVER REGULATION

Mathematical Models

Potentially mathematical modeling procedures offer the best solution to the problem of predicting channel response to river regulation. This type of model has been successfully used to predict bed scour and deposition in response to changes in discharge and sediment load. Routing procedures based on the continuity, flow resistance, and sediment transport equations are used to determine the volume of sediment transported through a series of channel reaches in a given time interval. Changes in bed elevation and bed material size enable the flow geometry and sediment transport to be determined in the succeeding time interval (U.S. Army Corps of Engineers 1977).

Unfortunately existing mathematical models are limited in their application and further research is required on bank erosion/deposition and meander processes before it will be possible to predict changes in channel width, plan shape, and valley slope. Even when applied to predict scour and fill in gravel-bed rivers inaccuracies can result due to the deficiencies of the flow resistance and sediment transport equations used in the model. Bray (in press) has shown that the Strickler type flow resistance equations can produce an error of 40% in the predicted velocity, while the entrainment function for graded gravel material can be as low as 0.03% (Neill 1968).

Empirical Models

Until mathematical models have been satisfactorily developed recourse has to be made to much simpler empirical methods for prediction purposes. Regime equations can give an indication of the type of channel response to river regulation but they cannot be used to predict the new equilibrium hydraulic geometry of the channel. Feedback mechanisms enable the channel to adjust its own dominant dicharge, sediment load, and bed material size and these values cannot be predicted in advance. However simple field methods can be used to establish the general response of the channel to regulation.

The first requirement is to investigate the natural stability of the river as this enables the sections of channel which will be most susceptible to changes in the flow regime to be identified. The next stage involves the determination of the critical threshold discharges for bed material transport and bank erosion at a number of representative sections along the river. This enables an upper limit to be established for regulated flows which will preserve the natural stability of the channel downstream from any proposed intake/outfall or dam site. In addition it enables the sections of channel which will require maintenance and protection to be identified if regulated flows are required above the threshold discharges for bed material transport and bank erosion.

Natural Channel Stability. Unstable sections of channel can be identified in a variety of ways. Direct evidence of erosional and depositional activity, or evidence of systematic changes in the cross section or plan shape of the river over a number of years are the best indicators. However if this type of information is unavailable it is possible to make an assessment based on the morphological or hydrological characteristics of the river.

Several common morphological features develop as a channel responds to erosional and depositional activity. During erosion the river incises into the bed of the channel and this produces relatively deep channel sections, steep gradients, large bed material, and high average flow velocities. In contrast rivers that are actively depositing produce relatively wide shallow channels with low gradients, small bed material, and low average flow velocities.

These systematic changes in channel capacities have important repercussions on the frequency of overbank flooding. In gravel-bed rivers the return period (annual series) of bankfull flow for eroding sites is greater than 1.5 years while for depositing sites it is less than 1.5 years.

Bed Material Transport Thresholds. Threshold discharges for bed material transport can be obtained either from rating curves, from calculations based on sediment entrainment functions or by direct observations of bed material tracers during flow events of varying magnitudes.

Although rating curves offer the simplest solution to the problem, in practice insufficient sites are calibrated, especially in gravel-bed channels.

For sand-bed channels the Shields entrainment function can be used to establish threshold discharges for movement given hydraulic data for various flow levels and information on the grain size frequency distribution of the bed sediment. Considerable error can arise when this method is applied to gravel-bed channels because transport thresholds are significantly affected by the degree of imbrication and the packing of the gravel.

Field experiments with bed material tracers offer the best solution to the problem of assessing threshold discharges for sediment transport in gravel-bed rivers. At each site the bed material size distribution is defined by sampling and measuring the intermediate axis of a hundred pebbles obtained from the bed of the channel using a grid sampling procedure. This sample is divided and half of the pebbles are painted and replaced in a line across the channel perpendicular to the banks. On practical grounds material less than 10 mm is not used for tracing purposes. In addition, as tracers are unlikely to be replaced in a natural position, some movement is expected at flows below the actual transport threshold until they become re-established on the bed of the channel. Any movement less than 0.5 m is considered to be due to this process and is disregarded in the analysis.

Observations of the tracers during and after various known flow events enables the maximum peak flow which does not cause movement, and the minimum peak flow which causes movement, to be defined. Given a favorable range of flow conditions these values converge and the threshold discharge for bed material movement can readily be defined. If the peak flows do not range around the threshold discharge during the period of the experiment it is necessary to estimate its probable value given the two limiting flows.

Bank Erosion Thresholds. To assess the limiting threshold discharge for bank erosion several sites which display fresh evidence of bank collapse need to be instrumented. At each site 1 m long steel reinforcing rods are driven horizontally into the coarse material underlying the bank at approximately 4 m centers along the length of the eroding bank. Any subsequent exposure of the rod indicates the amount of gravel removed. To monitor erosion of the overlying cohesive sediment a line of reinforcing rods is driven vertically into the surface of the flood plain exactly 1 m from the edge of the bank and at approximately 4 m centers. This serves as a datum for resurveying purposes. Regular site visits enable the threshold discharge for bank erosion to be established.

Channel Response to Regulation. Given information on threshold discharges for bed material transport and bank erosion at a number of representative locations along a river, it is possible to define a regulation strategy which will maintain the natural stability of a channel downstream from any proposed regulation point. Provided natural flows do not increase significantly downstream when regulation occurs, the maximum regulated flow at the dam site or river intake/outfall is defined by the minimum threshold discharge for bed material transport or bank erosion downstream from the regulation point. However, if natural flows increase rapidly downstream during periods of regulation, the maximum regulated flow at the dam site or river intake/outfall is defined by the threshold discharge for bed material transport or bank erosion where instability would first occur minus the downstream increase in base flow.

If regulation alters the frequency of flows which can transport sediment, instability will result. Erosion will take place at any point where regulation increases the frequency of flows above the threshold discharge for bed material transport and bank erosion. The degree of incision will largely be governed by the occurrence of bed rock ledges or the extent to which the bed becomes armored by coarser material. Severe degradation can promote the upstream migration of erosional activity and this will cease only after the channel has adjusted its hydraulic geometry and bed sediment size until all regulated flows are below sediment transport thresholds. Bed rock outcrops restrict headward erosion but they cannot prevent scouring further downstream. Material derived from these reaches will be deposited lower downstream at sections where maximum regulated flows are below transport thresholds. If depositional activity is persistent this may promote aggradation further upstream.

Any erosion which occurs at an intake/outfall point will promote upstream degradation unless this is balanced by aggradation during periods of abstraction. Although it is theoretically possible to maintain a dynamic stability by this process, in practice this is unlikely due to changes in channel width and plan shape during periods of erosion which cannot be reversed. By locating the intake/outfall in a section of channel containing large boulders or bedrock, or by constructing an artificial ledge across the channel it is possible to prevent erosional activity progressing upstream. However aggradation can still occur unless abstractions take place at flows below transport thresholds.

Simply by comparing regulated flows with threshold discharges for transport and erosion at a number of locations it is possible to identify which sections of channel will require maintenance and protection. While it is not currently possible to predict the new hydraulic geometry of the potentially unstable reaches for reconstruction purposes, guidance can be given on the nature of the necessary protection works or maintenance procedures to minimize the effects of instability. In alluvial sections where erosion is predicted it will be necessary to protect the channel banks because the river will tend to reduce its gradient by incision and meandering. Artificial barriers or drop structures will also need to be constructed to decrease channel slope over selected reaches. These could be designed to reduce the strength of the secondary flows which influence erosional activity. Where bedrock is exposed or where the bed will rapidly become armored by coarse material it may not be necessary to carry out any major protection or maintenance works. Sections of channel which will be subject to deposition can best be maintained by the systematic removal of sediment using earth excavators.

The methods outlined in this paper have been used to assess the effect of river regulation on the stability of the Rivers Wye, Severn and Dulas in the United Kingdom and to prescribe a regulation strategy which will preserve the natural stability of these rivers. The lack of bed material transport information in the United Kingdom precludes the use of any mathematical modeling procedures.

EFFECT OF PROPOSED DUTCH GULCH LAKE ON POPLAR CREEK

Reservoir operation will considerably modify the flow in Poplar Creek downstream from the proposed Dutch Gulch Lake. At the dam site the maximum release will be 12,590 cfs and the frequency of flows less than this value will be increased. The sediment discharge immediately below the dam site also be negligible because the trap efficiency of the reservoir will probably be 100%.

Unfortunately insufficient is known about all the governing equations for gravel-bed rivers to similate the effect of these changes on the hydraulic geometry of Poplar Creek. Equally it is not possible to identify exactly which reaches will undergo erosion or deposition as no information is available on threshold discharges for bed material transport. Ideally this information should be available for pool and riffle sections in reaches of channel which are naturally eroding, depositing, and stable. The available cross-sectional information and bed material data were insufficient to calculate these values. However, a sediment rating curve derived using the Meyer-Peter, Muller equation for a site 2.0 miles upstream from the dam and above the North Fork tributary predicts that flows of 500 cfs can transport 150 tons of bed load per day. Reference to the regulated daily flow duration curve indicates that bed material will be transported at least 20% of the time and probably as much as 50% of the time as this particular equation tends to underestimate transport rates. As this material will not be replenished erosion will take place immediately below the dam.

Qualitatively it is possible to assess the general effect of the reservoir on Poplar Creek on the basis of case studies (Kellerhals in press), geomorphological principles (Hey 1979, Schumm 1977) and regime equations (Bray, in press and Hey, in press). The amount of erosion below the dam partly depends on the rate at which the bed becomes armored by the coarser fraction of the bed material. If the banks were nonerodible the supply of sand would be limited and the bed would quickly become armored. However, as the banks are erodible there will be considerable amounts of sand available for transport and this will retard the armoring process.

Over the short time scale, less than 100 years, slight scouring will reduce the slope of the channel below the dam and increase the flow depth and bed material size. The reduction in flood flows will enable vegetation to colonize the exposed bars and this will tend to stabilize the point bars and probably promote increased bank erosion. Channel width will probably be reduced by this process. The sinuosity of the channel is likely to be increased slightly due to the reduction in channel slope and the pool/riffle spacing will also be reduced as this is controlled by the width of the channel. Erosion will continue below the dam site until releases can no longer transport bed material. The presence of bed rock outcrops would limit the degree of incision but there is no evidence for rock outcrops on Poplar Creek. Erosional processes will progress downstream in response to the progressive reduction in sediment load.

The downstream limit of erosional activity will probably be the confluence of Dry Creek, Little Dry Creek, and Poplar Creek. Deposition is likely to occur downstream from Little Dry Creek and Dry Creek due to reduced river levels in the main streams during flood events on the two tributaries. This reduction in river level could also promote erosion in the tributaries. Aggradation will probably progress upstream on Poplar Creek as material scoured from below the dam is deposited. This action will help to limit the amount of scour below the dam and also promote deposition in the tributaries. As a result of aggradation the slope and width of the channel will be increased while the depth and bed material size will be reduced. There will probably be a tendency for the channel to braid.

Simultaneously deposition will be occurring further downstream at the confluence of South Fork and Poplar Creek. Reduced flow stages on the main river could also promote erosion on South Fork.

In the longer [illegible] sediment yield of the tributaries is likely [illegible] deposition. The main

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stream will probably begin to erode the material deposited at the tributary confluences and a new, less vigorous, scour/fill cycle will be initiated.

The projected net changes for short and long time periods are summarized in Table 1.

CONCLUSIONS

In spite of the deficiencies of the data base, particularly the lack of information on the hydraulic geometry and associated bed material at pool/riffle sections in the naturally eroding, depositing, and stable reaches, the absence of bed material transport data and the non availability of historical maps and aerial photographs, it is possible to assess the general response of Poplar Creek to the construction of Dutch Gulch Lake. Quantitative estimates of the degree and rate of channel response to regulation are dependent on the availability of more precise survey data.

Even if the requisite data were available it is doubtful whether there would have been complete agreement between the various modeling methods on the predicted outcome. Moreover there is no way in which the various modeling techniques can be assessed. This clearly identifies the need for basic information on the systematic changes in the hydraulic geometry and sedimentary characteristics of rivers affected by regulation so that existing predictive models can be evaluated or new more appropriate modeling procedures developed.

Table 1. Predicted short and long term response of Poplar Creek to construction of Dutch Gulch Lake.

	POPLAR CREEK (Dam to Little Dry Creek)		POPLAR CREEK (Downstream Dry and Little Dry Creeks)		DRY AND LITTLE DRY (Downstream Sections)	
	less than 100 years	greater than 100 years	less than 100 years	greater than 100 years	less than 100 years	greater than 100 years
Nature of Instability	E	S	D	E	E	D
Slope	-	0	+	-	-	+
Width	-	0	+	-	-	+
Depth	+	0	-	+	+	-
Bed Material Size	+	0	-	+	+	-
Pool/Riffle Spacing	-	0	} BRAID	-	-	} BRAID
Sinuosity	+	0		+	+	

+ increase; - decrease; 0 no further significant change; D deposition; E erosion; S stablizes.

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A PREDICTION OF THE DOWNSTREAM RESPONSE OF POPLAR CREEK, CALIFORNIA TO THE DUTCH GULCH DAM

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INTRODUCTION

The present paper represents my effort to predict the likely hydraulic and morphologic response of Poplar Creek downstream of the proposed Dutch Gulch Dam to the extent that is allowed by the voluminous but incomplete data provided to me by the workshop organizers.

If the reader considers the use of the first person pretentious, allow me to remark that a prediction of this sort is by nature a highly personal one. "Intelligent" guessing is required in step after step in order to fill the gaps in even the most comprehensive data set. By using the first person, I hope to elucidate the thought processes of the subjective decisions necessary for this prediction.

The prediction proceeds in 11 steps outlined below.

1. Delineation of the reaches.
2. Geometric data.
3. Bed material.
4. Geometric data synthesized from hydrologic considerations.
5. Natural and project flow duration curves.
6. Water and gravel routing for natural conditions.
7. Short term water and gravel routing for project conditions ignoring tributary degradation.
8. The short term effect of tributary degradation.
9. Long term changes in channel geometry.
10. Meandering and the pool-and-riffle structure.
11. Summary of predictions for "Poplar" Creek and its tributaries.

DELINEATION OF THE REACHES

I considered the four reaches schematized in Figure 1. They are:

Reach 1. Poplar Creek from Dutch Gulch to just upstream of Dry Creek.

Reach 2. Poplar Creek from just upstream of Dry Creek to elevation 430 feet.

Reach 3. Little Dry Creek from the confluence with Poplar Creek to elevation 540 feet.

Reach 4. Dry Creek from the confluence with Poplar Creek to elevation 560 feet.

The elevations refer to stream elevations on the 1965 U.S. Geological Survey 1:24000 map of the Hooker, California quadrangle. The 430-foot point is about 2 km upstream of the confluence with the South Fork.

In fact, Dry Creek and Little Dry Creek join Poplar Creek at points so close together that for the purposes of the analysis I decided to proceed under the assumption that the confluences coincide at a point corresponding to the average of the two.

The reason for the above choice of reaches is as follows. Having been given practically no information about the South Fork or any projects on it, I assumed that it remains unchanged. To be on the safe side, Reach 2 was chosen so as not to include it. There are no tributaries of any consequence in Reach 1, which would be the one immediately below the dam. Reach 2 contains two tributaries of consequence; namely, Reaches 3 and 4 at its upstream end.

Both water and gravel can be routed through the system of four reaches for natural and project conditions. Down-channel reach lengths are given in Table 1.

Table 1. Down-channel reach lengths.

REACH	LENGTH (km)
1	8.26
2	6.23
3	6.13
4	4.57

MEASURED GEOMETRIC DATA

I determined overall down-channel slopes S for the four reaches by considering the elevation drop over lengths chosen to contain, but be

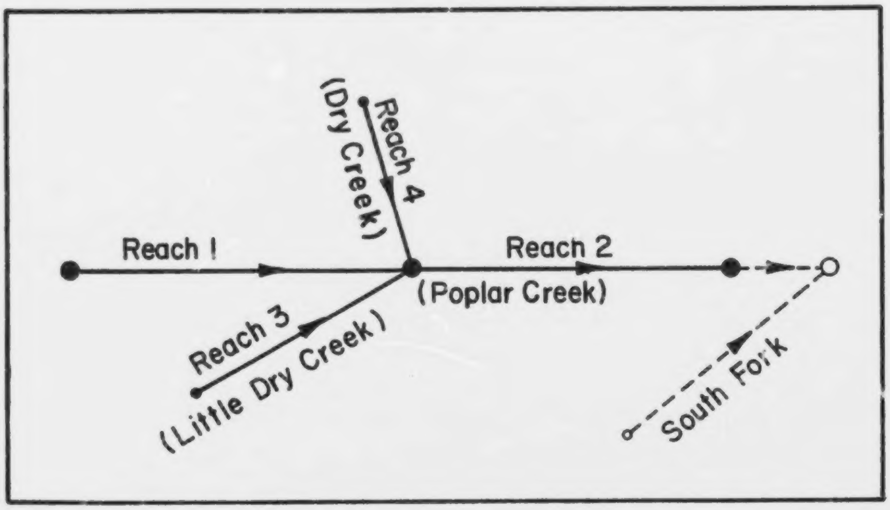

Figure 1. Definition of reaches.

a little longer than, each reach. The values were read from the supplied 1:24000 topographical maps. The values are $S = 0.00180$, 0.00186, 0.00390, and 0.00569 for Reaches 1 to 4, respectively.

Data on channel cross-sectional geometry were supplied only for Reach 2. I determined average bankfull water surface width B_b and cross-sectionally averaged depth H_b for the cross sections labeled 57 to 65. The values are $B_b = 119$ m and $H_b = 2.25$ m.

Values for B_b and H_b for the other three reaches were synthesized in the fashions outlined subsequently.

BED MATERIAL

The bed material data on Poplar Creek dated August 1978 were used to obtain a representative grain size distribution. Of the seventeen samples listed therein, I could not locate samples 2F-78-3, -4, or -5 on the attached map. Of the remaining fourteen samples, seven were taken at the water's edge, and seven were from overbank. Each sample is a bulk sample of at least two sacks.

Most gravel-bed streams have a surface pavement which is two or three times coarser than the underlying subpavement. A large bulk sample taken in the submerged channel typically contains mostly subpavement, but considerable amounts of pavement. However, Parker and Klingeman (1980) have argued that large bulk samples taken on exposed bars near the water's edge provide a reasonable approximation of the subpavement of the channel proper. I used this assumption in analyzing the seven data sets from the water's edge.

The sample 2F-78-1 proved to be markedly finer than the six others, so I rejected it as being uncharacteristic. The other six samples were averaged to yield a "typical" expected subpavement distribution for Poplar Creek, including Reaches 1 and 2. The average size distribution is given in Figure 2. Subpavement D_{50} and D_{90} are seen to be about 20 and 106 mm, respectively; fines (sand) content is about twenty percent.

None of the seven overbank samples appeared to be anomalous. I averaged them to obtain a "typical" overbank sediment distribution with $D_{50} = 25$ mm, $D_{90} = 130$ mm, and eighteen percent fines content. The average overbank size distribution is also known in Figure 2.

Several relevant observations can be made here. The values of subpavement D_{50}, D_{90}, and fines content for Poplar Creek are very similar to the values for four gravel-bed streams on which extensive gravel bedload measurements have been made; namely Oak Creek, Oregon; Elbow River, Alberta; Vedder River, British Columbia; and Snake River,

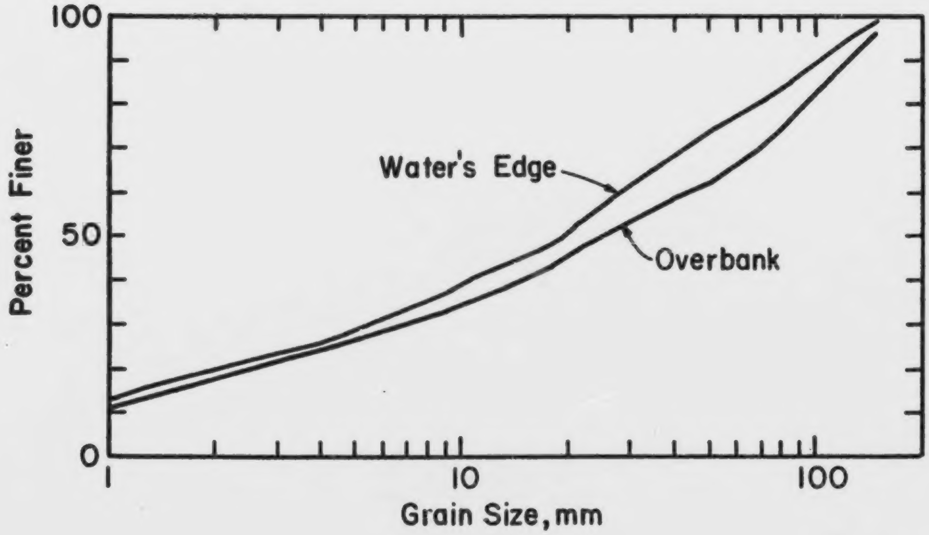

Figure 2. Grain-size distributions.

Idaho. Parker, Klingeman, and McLean (1980) have used these data to determine a bedload relation for field gravel-bed streams. Thus, I feel at least partially justified in applying the relation to Poplar Creek.

Also, the overbank size distribution is about the same as that of the subpavement. This suggests that vegetation and cohesive material do not act as significant controls on channel width. Recently I (Parker, 1980) determined several sets of empirical relations for bankfull hydraulic geometry. One set, using 21 reaches mostly from Alberta and British Columbia, Canada corresponds to gravel-bed streams with gravelly banks and little cohesion, in regions that are semiarid or only moderately humid. The width and depth relations are shown in Figures 3a and 3b; they and the corresponding correlation coefficients are:

$$B_b = 5.86\ Q_b^{0.441} \qquad r^2 = 0.931 \tag{1a}$$

$$H_b = 0.188\ Q_b^{0.416} \qquad r^2 = 0.860 \tag{1b}$$

where Q_b is bankfull discharge. The units are S.I. I assumed these relations to hold with a reasonable degree of accuracy for Poplar Creek and its tributaries.

A third point concerns roughness height k in the Keulegan resistance equation; where V is flow velocity and g is gravitational acceleration,

$$\frac{V}{\sqrt{gHS}} = 2.5\ell n\ (11\frac{H}{k}), \tag{2}$$

Parker and Peterson (1980) have followed the lead of several other investigators and have provided justification for the approximation

$$k = 2D_{p90}$$

at flood stages (the only stages which normally move gravel in gravel-bed streams). The subscript p refers to pavement.

Pavement size distributions for Poplar Creek were not available. However, experience suggests to me that a bulk surficial median pavement size should be two to three times coarser than the subpavement D_{50}; i.e.

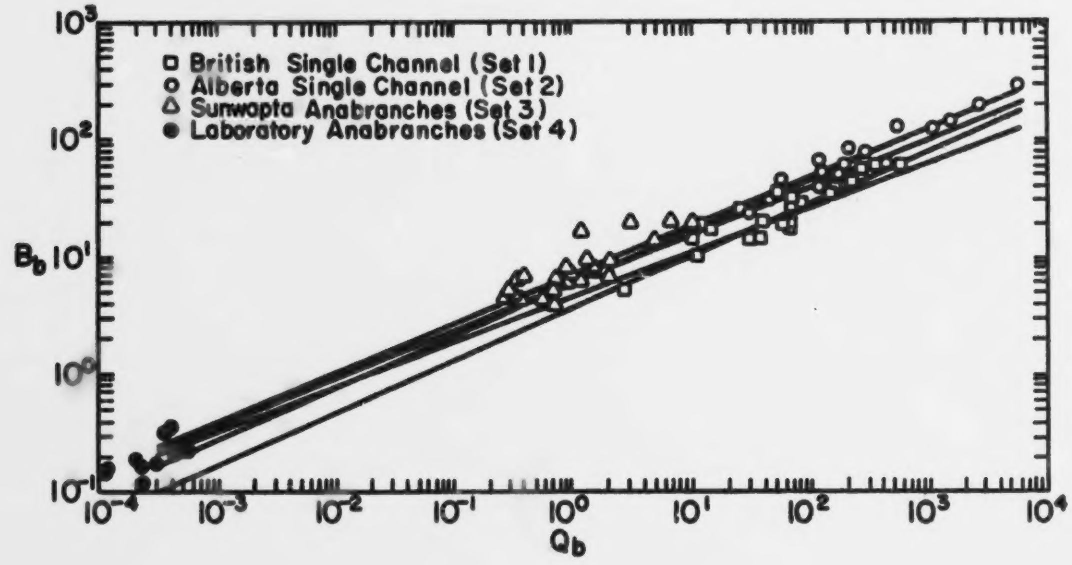

Figure 3a. Width relations.

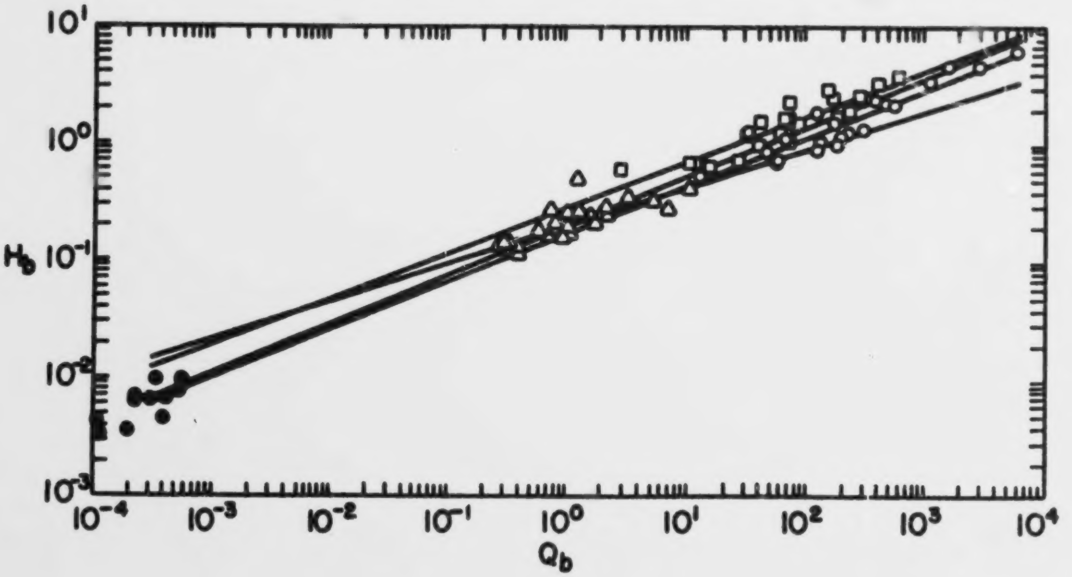

Figure 3b. Depth relations.

in this case $D_{p50} \simeq 50$ mm. However, the same experience suggests that pavement D_{p90} and subpavement D_{90} are in most cases not much different. With this in mind, I estimated $k = 0.212$ m.

Available data for Reaches 3 and 4 were at best sketchy. The only bed samples pertaining to them were hole number 5 for Little Dry Creek, with a D_{50} of 10 mm, and hole number 1 for Dry Creek, with a D_{50} of 20 mm. Under these circumstances, and considering that all four reaches receive their gravel from adjoining areas which are likely to have similar lithology and weathering, I decided to assume that subpavement D_{50} is 20 mm and roughness $k = 212$ mm for all four reaches.

GEOMETRIC DATA SYNTHESIZED FROM HYDROLOGIC CONSIDERATIONS

Equation 2 can be used to relate discharge to geometric parameters at flood stages in wide channels.

$$Q = 2.5 \sqrt{gS}\, H^{3/2} B \ell n \left(11\frac{H}{k}\right) \tag{3}$$

I used this equation to predict bankfull discharge Q_b in Reach 2, using the previously determined values of k, H_b, B_b, and S. The predicted value of Q_b is 646 cumecs.

In order to check this value, I back-calculated Q_b from each of Equations 1a and 1b; the average of the two values determined thus is 656 cumecs. In addition, an evaluation of the flood frequency of this discharge (outlined subsequently) indicated that a peak flow of 646 cumecs corresponds to a return period of 3.6 years. A typical value for Alberta streams that are incised is about five years; in more humid regions the value drops to about two years. Thus the estimated value of Q_b for Reach 2, whether right or wrong, is at least reasonable.

The hydrologic data include monthly flows for Poplar Creek near Dutch Gulch, (i.e., Reach 1) the South Fork, and Poplar Creek near Cottonwood. Also provided are flood frequency curves for Poplar Creek near Dutch Gulch and Poplar Creek near Cottonwood. There are only four tributaries of consequence from Dutch Gulch to the gaging station at Cottonwood; they are in order downstream: Dry Creek, Little Dry Creek, South Fork, and Hooker Creek. Thus I assumed the following hydrologic algorithm.

$$Q_{DG} + Q_{DC} + Q_{LDC} + Q_{SF} + Q_{HC} = Q_C$$

Q_{DG} = discharge in Poplar Creek at Dutch Gulch (Reach 1)

Q_{DC} = discharge from Dry Creek

Q_{LDC} = discharge from Little Dry Creek

Q_{SF} = discharge from South Fork

Q_{HC} = discharge from Hooker Creek

Q_C = discharge in Poplar Creek near Cottonwood

Insofar as gravel-bed streams usually only move gravel at flood stages, I concerned myself only with high flows. The flood frequency curves for natural conditions indicate that

$$Q_{DG} = 0.556\ Q_C \tag{5}$$

at both the two- and five-year floods. The monthly flows indicate that the river is highest for the months of January, February, and March. The average natural monthly flows for these three months obey the relations

$$Q_{DG} = 0.537\ Q_C \tag{6a}$$

$$Q_{SF} = 0.262\ Q_C \tag{6b}$$

Note the good correspondence between Equations 5 and 6a. After some adjustment, I finally adopted the relations

$$Q_{DG} = 0.556\ Q_C;\ Q_{SF} = 0.271\ Q_C \tag{7}$$

for flood stages.

It is seen from Equations 4 and 7 that typically 17.3 percent of the flood discharge observed at Cottonwood must originate from Dry Creek, Little Dry Creek, and Hooker Creek. Any estimate for partitioning flood flows among them based on the data provided to me must of necessity be very crude. I thus rowed with the oars I had. I measured basin areas A_{DC}, A_{LDC}, and A_{HC} from the topographical maps provided; the map of normal annual precipitation allowed me to estimate annual precipitations P_{DC}, P_{LDC}, and P_{HC}. The values are listed in Table 2.

Table 2. Annual precipitation values.

	BASIN AREA (km^2)	ANNUAL PRECIPITATION (mm)
Dry Creek	71	740
Little Dry Creek	154	700
Hooker Creek	83	600

Now let $r_{DC} = Q_{DC}/Q_C$ be the ratio of discharge from Dry Creek to the discharge measured in Poplar Creek at high flows under natural conditions: r_{LDC} and r_{HC} are assumed to be similarly defined. Note that from Equations 4 and 7,

$$r_{DC} + r_{LDC} + r_{HC} = 0.173$$

I related the values of r to the ratios of annual volumes of water falling on each of the basins; to wit

$$r_{DC} \{ \frac{A_{DC} P_{DC}}{A_{DC} P_{DC} + A_{LDC} P_{LDC} + A_{HC} P_{HC}} \} 0.173$$

and likewise for r_{LDC} and r_{HC}.

The results of this computation are:

$$Q_{DC} = 0.0433 Q_C \tag{9a}$$

$$Q_{LDC} = 0.0887 Q_C \tag{9b}$$

$$Q_{HC} = 0.0410 Q_C \tag{9c}$$

Now since the gaging station at Dutch Gulch is within Reach 1, it is appropriate to define $Q_{DC} = Q_1$. Likewise, one may define $Q_{LDC} = Q_3$ and $Q_{DC} = Q_4$. From Equations 7 and 9, these definitions,

$$Q_3 = 0.160 Q_1 \tag{10a}$$

$$Q_4 = 0.078 Q_1 \tag{10b}$$

By definition, the discharge in Reach 2, Q_2, is given by the algorithm

$$Q_2 = Q_1 + Q_3 + Q_4$$

or thus

$$Q_2 = 1.238\ Q_1 \tag{10c}$$

Equations 10 are assumed to hold for high natural flows. They involve the very drastic assumption that the tributaries are always perfectly in phase with the main stem. The total basin for Poplar Creek is not large, and both annual rainfall patterns and the rainfall pattern for the storm of January 14-18, 1974 (assumed to be fairly typical) do not suggest the tributaries to be strongly out of phase with the main stem. Dry Creek and Little Dry Creek have considerably smaller basins than Poplar Creek upstream of Dutch Gulch, and so would normally be flashier. However, even this is mitigated by the fact that the upper reaches of the Poplar Creek basin typically receive much more rainfall than the two tributary basins; the lag time for infiltration in the typically drier tributary basins may counteract their flashiness.

Now if bankfull discharge for Reach 2 is taken to be 646 cumecs, Equation 10c indicates a bankfull discharge for Reach 1 of 521 cumecs, assuming the same flood frequency for both. The flood frequency curve for Poplar Creek at Dutch Gulch indicates a return period of 3.6 years for this discharge, as was mentioned previously.

Extending the assumption of equal bankfull flood frequency to the tributaries, it follows from Equations 10a and 10b that Q_b = 83.3 cumecs for Reach 3 (Little Dry Creek) and 42.6 cumecs for Reach 4 (Dry Creek).

Data on bankfull hydraulic geometry must now be synthesized for Reaches 1, 3, and 4. Of Equations 1a and 1b, the former has the higher coefficient of correlation; I decided to use it to estimate bankfull width B_b in Reaches 3 and 4, and then to estimate bankfull depth H_b from the friction relation, Equation 3. The values thus obtained are B_b = 41 m and H_b = 1.03 m for Reach 3, and B_b = 30.0 m and H_b = 0.74 m for Reach 4.

A word of caution is in order about these values. The empirical width-discharge relation is one of the most consistent relations of river mechanics. Widths estimated from it can be expected to be fairly accurate in most cases if an accurate value of Q_b is used. However, herein Q_b itself has been estimated for the tributaries in a very approximate fashion. The subsequently presented gravel bedload calculations are rather sensitive to variations in B_b, H_b, and Q_b. In addition, the 1:24000 topographical map of the [illegible], California quadrangle indicates that much of the valley of Reach 4 (Dry Creek) is filled with dredge tailings. It is possible that the stream channel itself is ill defined there.

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A value of B_b for Reach 1 can also be estimated from Equation 1a directly, but this would not recognize the fact that this reach is adjacent to another reach of the same stream for which B_b is known, i.e. Reach 2. It is thus useful to cast Equation 1a in the form

$$\frac{B_{b1}}{B_{b2}} = (\frac{Q_{b1}}{Q_{b2}})^{0.441} \tag{11}$$

so that the previously determined value of B_{b2} has a role in determining B_{b1}. Bankfull width for Reach 1 is thus found to be approximately 108 m. (If Equation 1a were used directly, the value would be fourteen percent smaller.) Bankfull depth can then be estimated from Equation 13 as 2.12 m.

The values of B_b and H_b for Reach 1 are later adjusted slightly to facilitate gravel routing.

The measured and estimated bankfull parameters, and several others, for the four reaches under natural conditions are summarized in Table 3. L denotes the length of each reach, and B_G denotes the typical width of mobile gravel on the bed during floods sufficient to mobilize the bed. B_G has been estimated as

$$B_G = B_b - 10H_b \tag{12}$$

based on information on the cross sections and Parker (1979).

At this point, I questioned whether or not "bankfull" is even a valid concept for Reaches 1 to 4. Many gravel rivers are incised and have little or no genetic floodplain area. Both the cross sections and the topographic maps supplied indicate that Poplar Creek does have a floodplain in Reaches 1 and 2. The topographic map suggests that Little Dry Creek has a floodplain in most of Reach 3. The status of Dry Creek in Reach 4 is, however, open to question. Its valley seems to have been filled with dredge tailings. Whether this aggradational surface acts as a "surrogate floodplain," or whether the stream has incised into the deposits, remains unclear.

Table 3. Measured and estimated bankfull parameters, and several others, under natural conditions.

	REACH 1	REACH 2	REACH 3	REACH 4
Q_b (m^3/s)	521	646	83.3	40.6
H_b (m)	2.12*	2.25	1.03	0.74
B_b (m)	108*	119	41	30
B_G (m)	87	96	30.7	22.6
S	0.00180	0.00186	0.00390	0.00569
D_{50} (mm)	20	20	20	20
k (mm)	212	212	212	212
L (mm)	8.26	6.23	6.13	4.57

*subject to later adjustment

NATURAL AND PROJECT FLOW DURATION CURVES

The flow duration curves for both natural and project conditions at Dutch Gulch (Reach 1) were provided to me by the workshop organizers.

For a gravel routing scheme it is necessary only to consider flood flows high enough to break the pavement and mobilize the bed gravel. Parker and Klingeman (1980) have found a criterion due to Neill (1968) for bed motion in terms of pavement, namely

$$\tau_p = \frac{HS}{1.65D_{p50}} \geq 0.03$$

to take the approximate form in terms of subpavement

$$\tau^* = \frac{HS}{1.65D_{50}} \geq 0.0742 \qquad (13)$$

for many paved gravel-bed streams. (The implication is that D_{p50}/D_{50} is roughly 2.5.)

I used criterion (13), Equation 3, and an at-a-station relation for water surface width as a function of discharge in gravel-bed streams due to Parker and Peterson (1980) to estimate values of H and Q required for breaking the pavement. The at-a-station relation is

$$\frac{B}{B_b} = \left(\frac{Q}{Q_b}\right)^{0.16} \tag{14}$$

The value of Q so determined for each reach was reduced to an equivalent discharge Q_{e1} in Reach 1 via Equation 10. A frequency of exceedance was then determined from the natural flow frequency curve at Dutch Gulch, and converted to a number of days per year for which each reach could be expected to have a mobile gravel bed. The results are given in Table 4.

Table 4. Conditions for bed gravel mobilization.

	REACH 1	REACH 2	REACH 3	REACH 4
H (m)	1.36	1.32	0.63	0.43
Q (m^3/s)	210	215	29.5	13.0
Q_{e1} (m^3/s)	210	174	184	167
days exceeded per year	1.43	2.31	2.02	2.56

These values suggest that when applying the Dutch Gulch flow duration curve for natural conditions to gravel routing, flows less than 5912 cfs (142 cumecs) need not be considered. This corresponds to between a 1.1 and 1.2 year peak flood in Reach 1.

The portion of the Dutch Gulch flow duration curve relevant to gravel routing can then be converted to a histogram of discharge intervals $i = 1, 2, 3, \ldots$, typical discharge Q_i (geometric mean) on each interval, and the fraction of time per year p_i at which discharge falls in the interval. This is given for Dutch Gulch (Reach 1) for both natural and project conditions in Table 5: $(Q_i)_N$ refers to natural values, and $(Q_i)_P$ to project values in cumecs.

Thus, Table 5 constitutes the natural and project flow frequency relations for Reach 1. In accordance with the assumption that Reaches 1-4 are all perfectly in phase under natural conditions, the corresponding natural relations for Reaches 2, 3, and 4 can be obtained from Equation 10 by multiplying $(Q_i)_N$ by 1.238, 0.160, and 0.078, respectively, without altering the corresponding value of p_i.

Table 5. Discharge intervals, typical discharge, and fraction of time per year discharge fall in that interval for natural and project conditions.

INTERVAL NO.	p_i	$(Q_i)_N$	$(Q_i)_P$
1	0.0038	159	141
2	0.0028	200	192
3	0.0016	252	244
4	0.0009	318	277
5	0.0003	400	307
6	0.0001	503	328
7	0.0001	634	339
8	0.0001	798	350

The project flow frequency relations for Reaches 3 and 4 can be taken to be approximately the same as the natural ones, as the only effects of a dam at Dutch Gulch on these tributaries would be backwater effects due to lowered baseline. The project relation for Reach 2 is rather more difficult to synthesize.

In the absence of any other guidelines, I made a sweeping assumption that I know is often erroneous. I assumed that the dam is operated only so as to chop off the peaks of flood flows more or less uniformly, the water thus stored being released at low flow or lost to evaporation or use. If this is the case, then the system of Reaches 1-4 are approximately in phase even under project conditions.

Before proceeding on this assumption, it may help to analyze how it might lead to error. Suppose the dam is operated so as to completely eliminate rather than just lower, say, spring snowmelt floods. In the spring flooding, Reaches 3 and 4 flow into a main stem that is not in flood. In this case, the tributary baseline is much lower than if flood peaks had simply been lowered. Tributary degradation and head-cutting should occur in either case, but these processes would be much more severe when the main stem and tributaries are significantly out of phase.

Proceeding ahead boldly or foolishly as the case may be, perfect phasing under project conditions allows one to write

$$(Q_2)_P = (Q_1)_P + (Q_3)_N + (Q_4)_N = (Q_1)_P) + 0.238\ (Q_1)_N$$

where the subscripts 1, 2, 3, and 4 refer to Reaches 1-4, N and P stand for natural and project, and Equation 10 have been used. The project flow frequency relation for Reach 2 is then found from Table 5 by replacing $(Q_1)_N$ with $(Q_1)_P) + 0.238\ (Q_1)_N$, but leaving p_i unaltered.

WATER AND GRAVEL ROUTING FOR NATURAL CONDITIONS

The two basic tools for water and sediment routing are a resistance relation and a sediment transport relation. Herein Equation 3 is used for a resistance relation. The selection of a sediment transport relation, however, merits more care. Parker, Klingeman, and McLean (1980) have determined an empirical relation for bedload in paved gravel-bed streams based solely on field data. The relation is

$$W^* = 0.0025 \exp\left(14.2\ (\phi-1) - 9.28\ (\phi-1)^2\right) \qquad (15a)$$

where W^* is a dimensionless bedload and ϕ is a measure of relative stress;

$$W^* = \frac{Rq_B}{\sqrt{g}\ (HS)^{3/2}}$$

$$\phi = \frac{\tau^*}{\tau_r}$$

In the above relations $\tau^* = HS/RD_{50}$ is a Shields stress based on subpavement D_{50}, τ_r is a reference Shields stress equal to 0.0876 for field streams, q_B is volumetric bedload per unit width of bed gravel, and R is submerged sediment specific gravity.

Equation 15 is shown in Figure 4, along with field data used to determine it. Among the field streams listed on that figure, the Vedder River, British Columbia is quite similar to Poplar Creek. D_{50} is about 19 mm, D_{p50} is about 44 mm, D_{p90} is about 90 mm, and the subpavement contains about sixteen percent sand. S is near 0.00195, although it is affected by backwater from the Fraser River to a certain extent, and the ranges of width, depth, and discharge at which bedload has been measured are respectively, 85 – 90 m, 1.34 – 1.60 m, and 216 – 370 cumecs. In the context of computing bedload in Poplar Creek and its tributaries, this lends a glimmer of hope to an otherwise futile task.

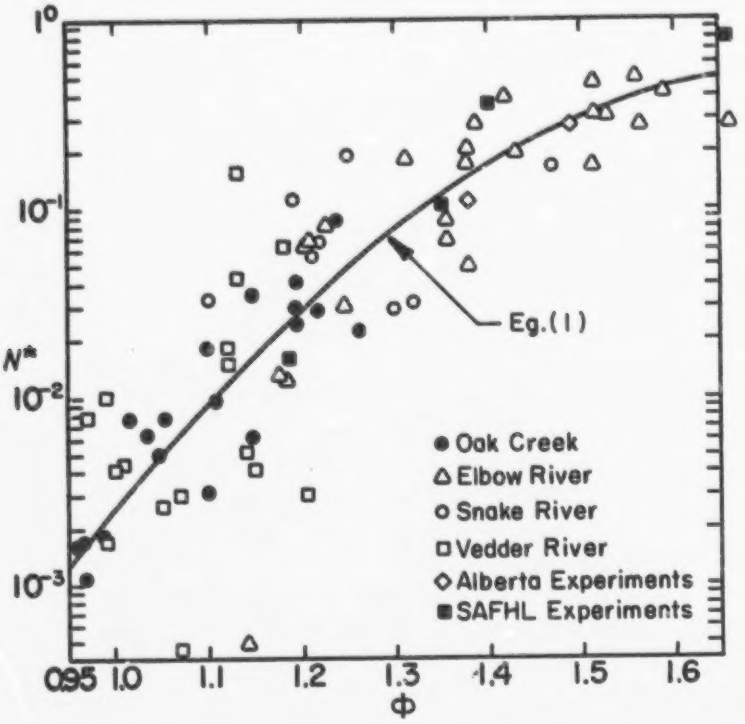

Figure 4. Gravel transport relation.

In fact, Equation 15a is valid only for $0.95 < \phi < 1.65$. For $\phi \geq 1.65$, the extension

$$W^* = 11.2 \left(1 - \frac{0.821}{\Phi}\right)^{4/5} \qquad (15b)$$

may be used. For $\phi < 0.95$ gravel loads are so small that W^* can be set equal to zero in a calculation of annual gravel yield.

It must be understood, however, that Equation 15 cannot predict total bedload in paved gravel-bed streams. Implicit in its incorporation of only a subpavement median grain size to describe grain size distribution is the assumption that the size distribution of the annual yield of bedload is similar to that of the bed material. This is in fact the case in many gravel-bed streams. Other such streams, however, may transport considerable quantities of sand as bedload over a gravel bed, so that the content of sand in the bedload far exceeds its bed content. The quantity of sand moving at any given time is a function of supply from outside sources. Thus sand moves through the stream without increasing sand content in the bed, and acts very much like wash load. As long as the bed material remains essentially a gravel matrix containing not more than about 25% sand in its pores (i.e. not exceeding the porosity of the gravel matrix), this and "throughput" load plays only a negligible role in the process of slow aggradation or degradation; thus the use of Equation 15, which neglects it, is justified.

There is one case, however, where it would appear that throughput sand can be incorporated in large quantities in the bed, namely, rapid deltaic aggradation at, e.g., the mouth of a tributary. The author has observed this in pilot laboratory experiments at the University of Alberta, and Milhous (1980) reports several field occurrences.

Let M_{Ni} $i = 1, 2, 3, 4$ denote this mean annual yield of gravel from Reaches 1 to 4, in metric tons, based on the natural flow frequency curve. Assuming that the stream system is in grade, it follows from the definitions in Figure 1 that

$$M_{N2} - (M_{N1} + M_{N4} + M_{N3}) = 0 \qquad (16)$$

In fact, there are enough vagaries in a natural system and uncertainties in such simple equations as Equations 3 and 15 so that predictions based on them rarely satisfy Equation 16 exactly. However, a sediment routing that does not predict graded conditions when conditions actually are in grade can hardly be expected to predict stream response to imposed changes accurately. I resolved the problem by "zeroing" the system. Herein this is accomplished by making slight changes in bankfull width of one of the reaches until Equation 16 is satisfied.

The calculation of annual gravel yield is performed as follows. Consider the ith reach at the jth discharge range of its flow duration curve, i.e. at discharge Q_{ij}. Width at this discharge is calculated from Equation 14 and depth from Equation 3. The only exception is that for which Q_{ij} exceeds Q_{bi}, the bankfull discharge. In this case, I made the facile assumption of infinite floodplain storage, so that "effective" depth and width are never allowed to exceed their bankfull values. Volumetric bedload per unit width of bed gravel $(q_B)_{ij}$ is calculated from Equation 15. The total annual mass yield M_i is then obtained from an appropriate summation in j:

$$M_i = 8.36 \times 10^7 \, B_{Gi} \sum_j (q_B)_{ij} P_{ij} \tag{17}$$

(The factor 3.15×10^7 converts seconds to years; a specific gravity of 2.65 is assumed for the gravel.)

Annual yields calculated in this fashion before zeroing are shown in Table 6. It is seen therein that

$$M_{N2} - (M_{N1} + M_{N4} + M_{N3}) = 97.0 \text{ tons/year}$$

providing a measure of the deviation from Equation 16. Trial-and-error adjustment of the bankfull width of either Reach 1 or Reach 2 could be performed in order to ensure that the terms on the left-hand-side of Equation 16 equal zero. However, it was obvious to me that any adjustment should be done on Reach 1; the value of B_b for it was synthesized, whereas the value for Reach 2 was measured.

The results of such an adjustment for Reach 1 are $B_b = 97$ m and $H_b = 2.26$ m (calculated via Equation 3); this amounts to a 10% reduction in bankfull width. The zeroed value of M_{N1} is shown in Table 6.

Table 6. Annual yields calculated before and after zeroing.

REACH	M_{Ni} (metric tons/year)
1 before zeroing	107.2
1 after zeroing	206.2
2	372.3
3	75.2
4	92.9

Equation 16 is seen to be satisfied within one percent. The low yields of Table 6 are, in my experience, not unusual for gravel-bed streams with self-formed gravel banks.

SHORT TERM WATER AND GRAVEL ROUTING FOR PROJECT CONDITIONS IGNORING TRIBUTARY DEGRADATION

Water depths and gravel yields may now similarly be calculated for project conditions using the flow duration curves developed in Section 5 and the zeroed values of B_b, H_b, and B_G for Reach 1. Herein this is done under the hypothesis (later found to be wrong) that gravel yields as well as water yields remain unaltered in the tributaries. That is, project yields M_{P3} and M_{P4} are assumed to equal, respectively, M_{N3} and M_{N4}.

Project gravel yields are shown in Table 7.

Table 7. Project gravel yields.

REACH	M_{Pi} (metric tons/year)	Percent of Natural Value
1	26.5	13
2	104.9	51
3	75.2	100
4	92.9	100

The predicted values apply for the short term (first few years).

It is seen that the effect of the dam on Reach 1 is profound; annual gravel yield which was not large under natural conditions has been reduced to only a small fraction of that. Although a detailed calculation using the method of Parker and Klingeman (1980) was not performed, the values of ϕ associated with project conditions in Equation 15 are small enough to indicate that the gravel load consists mostly of the finer grains available in the pavement, so that the pavement coarsens into an armor, and Reach 1 tends toward static equilibrium.

The coarsened surficial layer acts to protect the substrate from modification. Thus no immediate change in subpavement structure should occur. Indeed, the only long term modification likely to occur is the collection of fine material and organic debris from local sources in the substrate pores, as flows adequate to "flush" the bed are not likely to occur.

In Table 7, Reach 2 is also seen to have a lowered ability to move sediment under project conditions. The effect of the project is not so great due to the unaltered water supplies from the tributaries. On the other hand, the tributaries are feeding in sediment at an unaltered rate. The result is deposition of sediment at the upstream end of Reach 2 as the main stem is unable to move the contributions from the tributaries. The deposition rate is

$$M_{P1} + M_{P4} + M_{P3} - M_{P2} = 89.7 \text{ tons/year} \qquad (18)$$

Initially this should occur as a deltaic deposit at the mouths of the tributaries. Later, it is shown that this value vastly underestimates the short term deposition in Reach 2. However, it is seen that gravel continues to move through Reach 2 fairly actively. Except in the immediate vicinity of deltaic deposits, where fines may be trapped, both gravel payment and subpavement should remain relatively unmodified in structure, implying a healthy substrate for fish.

Short term degradation and aggradation rates can be estimated by the equation of conservation of bed sediment, where p is bed porosity, z is bed elevation, t is time, and x is a down-channel parameter,

$$\frac{\partial z}{\partial t} = \frac{1}{1-p} \frac{\partial q_B}{\partial x} \qquad (19)$$

Cast in a discrete form appropriate for the present calculation, it takes the form

$$\Delta \bar{z} = \frac{1}{(1-p)B_G L} (M_{IN} - M_{OUT}) \frac{1}{2.65} \qquad (20)$$

where Δz denotes the change in mean bed elevation on a reach in meters in one year and M_{IN} and M_{OUT} are annual gravel transport rates into and out of the reach in tons.

The computed values of Δz for Reaches 1 and 2 are exceedingly small, being -2.5×10^{-2} mm for the former and $+8.7 \times 10^{-2}$ mm for the latter (the minus sign indicates degradation); a value of p of 0.35 has been assumed. Part of the reason for the smallness of the values may be the long extent of the reaches over which they are computed; i.e. the grid may be too coarse. However, even if the degradation rate in Reach 1 is actually one hundred times larger, the implication is unaltered; flows are reduced so much in this reach that it very quickly reaches a state hardly removed from static stability. The short term potential for degradation here is exceedingly small.

The small aggradation rate predicted in Reach 2 is, however, probably very erroneous. The source of the error is in the assumption $M_{P3} = M_{N3}$ and $M_{P4} = M_{N4}$ for the tributaries. Controlled discharge releases from the dam imply lower water surface elevation in Reach 2 during floods. This implies an immediately lowered flood baseline for both of the tributaries as soon as the project is put into operation. The only way the tributaries can respond to this is by degrading their bed. Degradation should work its way upstream; gravel delivery rates to the main stem should temporarily (several years or more) increase, and much more deposition should occur in Reach 2 than that indicated by Equation 18.

The lowered baseline is illustrated in Figure 5 in terms of a probability of exceedance of given depth values (based on the flow duration curve) for natural and project conditions.

It is perhaps of value to note that the qualitative predictions that I have made herein are in agreement with the field observations of Kellerhals and Gill (1973) on the Peace River and its tributaries downstream of the W.A.C. Bennett Dam, British Columbia.

THE SHORT TERM EFFECT OF TRIBUTARY DEGRADATION

While it is not difficult to reach the conclusion that Dry Creek and Little Dry Creek will be subject to degradation due to lowered baseline during flood, prediction of the degradation and the associated yields of gravel is a different matter.

I have attempted to obtain simple results by means of a crude numerical model along the following lines.

a. A "dominant discharge" Q_d was evaluated for each tributary; it is defined such that if continued constantly for the portion of the year for which $\phi \geq 0.95$ (i.e., gravel load is not vanishing), it would transport the natural annual gravel load. The fraction of the year for which $\phi \geq 0.95$ is 0.0031 for both tributaries. A value $Q_d = 59.7$ cumecs was found for

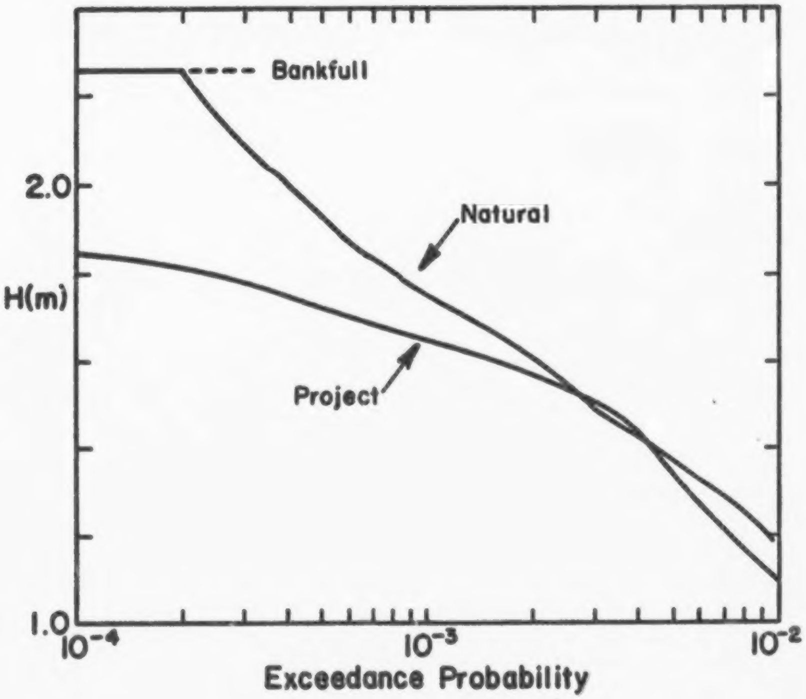

Figure 5. Depth exceedance curves, Reach 2.

Reach 3, and 28.8 cumecs for Reach 4. These correspond to discharges of 373 and 369 cumecs, respectively, in Reach 1 at the same exceedance probability of the flow duration curve, via Equation 10. A good average, then, is 371 cumecs on Reach 1, or via Equation 10, 459 cumecs on Reach 2. These discharges have an annual probability of exceedance of 0.000547, which is essentially identical to that of the dominant discharges in the tributaries. The concept of a yield-defined dominant discharge is introduced so that a degradation calculation can be performed under steady flow conditions that are in some way equivalent to a typical yearly hydrograph.

b. The depth in Reach 2 with a natural probability of exceedance of 0.000547 (i.e., the depth at 459 cumecs) is 1.90 m. The depth with the same probability of exceedance in the same reach under project conditions is 1.74 m (at a discharge of 386 cumecs). Thus a drop in baseline at "dominant" conditions of 0.16 m is realized.

c. This drop is realized in the short term in terms of a drop in main stem water surface level at the tributary mouth. A proper calculation of degradation thus requires backwater curves at each step. To avoid this in a simple calculation, I replaced decreased water surface elevation at the mouth with a step on the bed consisting of an initial drop of 0.16 m spread over 20 m from the tributary mouth upstream. I then assumed that normal depth calculated from Equation 3 is realized everywhere and at all times in the tributaries.

d. Each tributary was assumed to have the constant bankfull geometries listed in Table 3 from mouth (except for the step) to a point 2500 m upstream which was assumed to be far enough upstream not to be affected by degradation in the short term. Initial bed profiles are shown in Figure 6a. "Dominant discharge" was then imposed continuously on each tributary, and degradation was calculated by means of a numerical solution to Equation 19, with the aid of Equations 3, 14, and 15. One year of real time was assumed to pass for each period of 0.0031 years for which the numerical calculation was performed. Programming was done with the aid of the University of Alberta Amdahl computer.

The bed profiles of the tributaries before and after two years are shown in Figure 6b. It can be seen that 16 cm is gradually working its way upstream, and that considerable potential for degradation remains after two years.

In Figure 7 the cumulative sediment yield from each tributary is plotted versus time. After one year Little Dry Creek is seen to put 2550 tons of gravel into Reach 2, and Dry Creek yields 1670 tons. These values are, respectively, 34 and 18 times the annual natural yields.

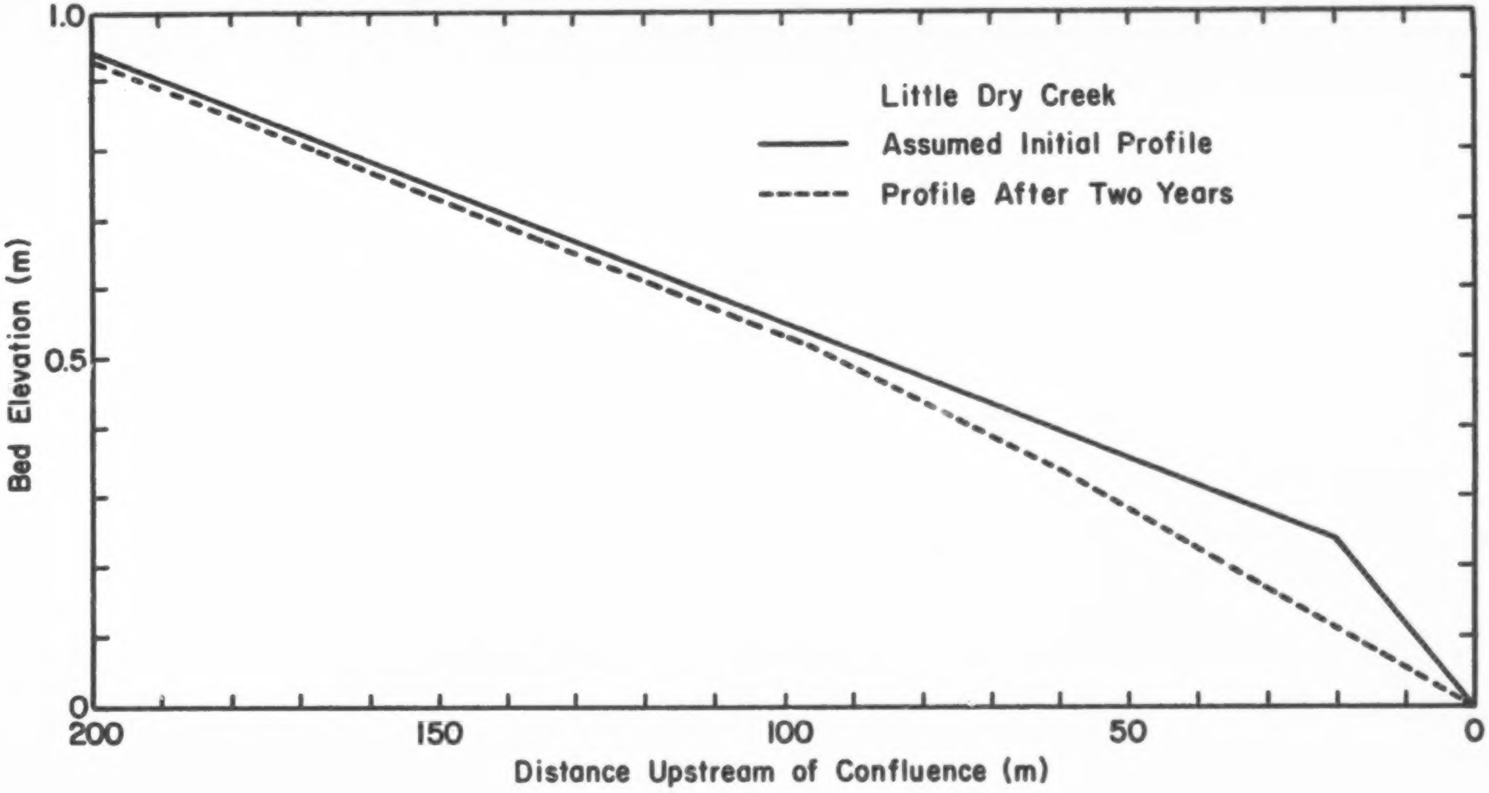

Figure 6a. Initial bed profiles.

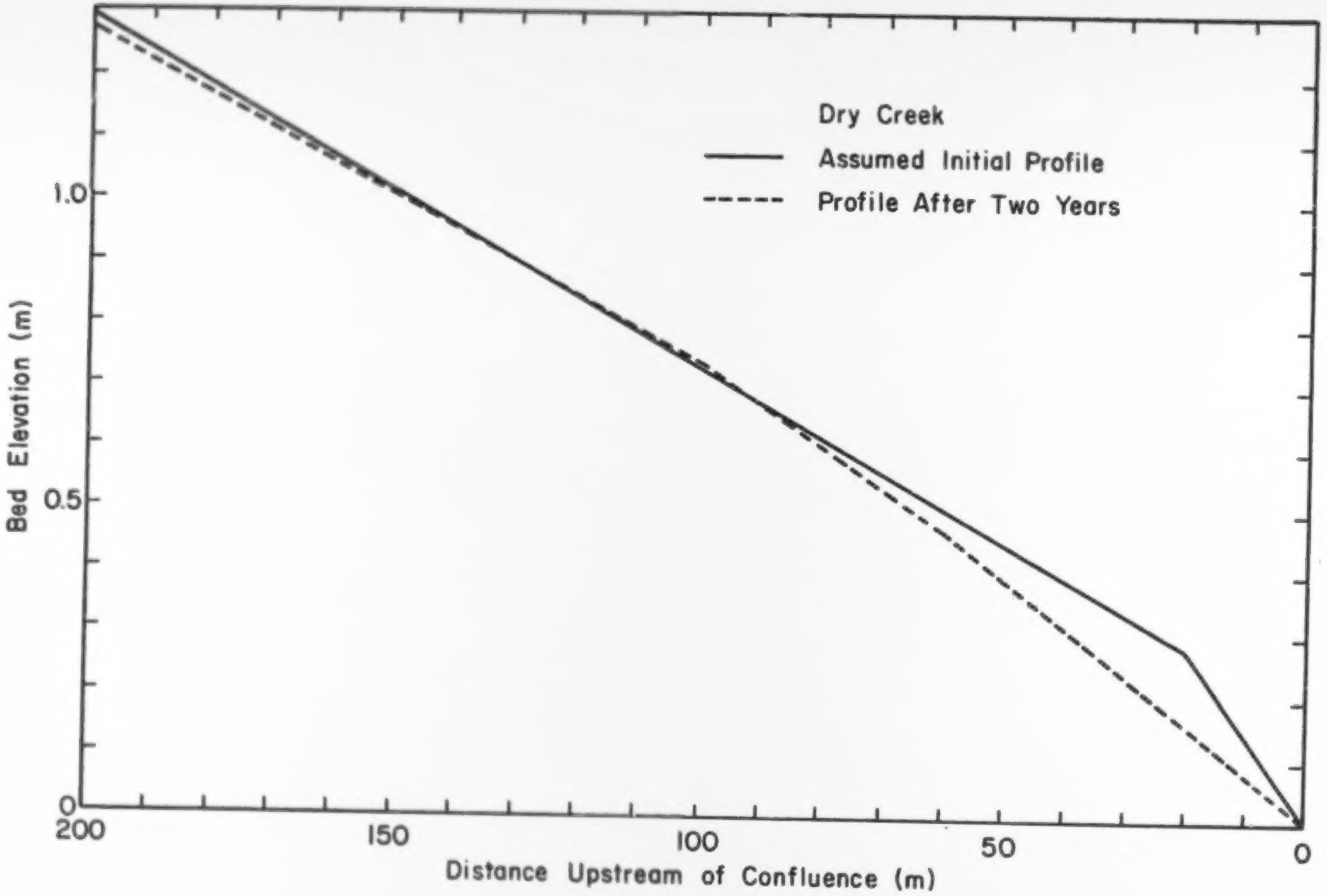

Figure 6b. Bed profiles after two years.

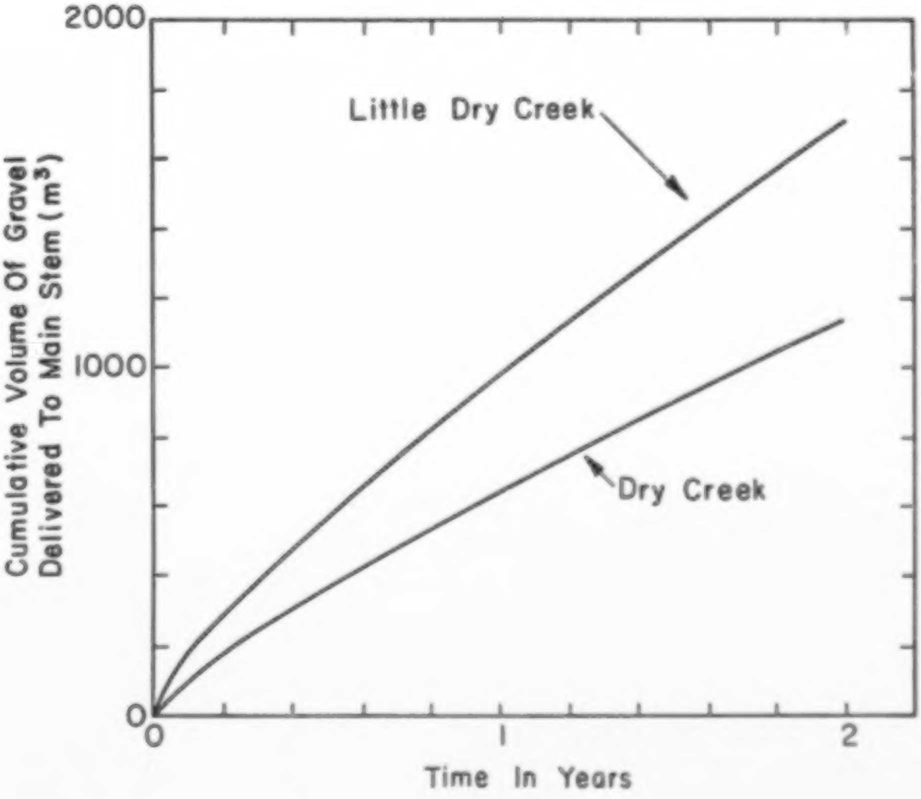

Figure 7. Gravel supply from tributaries.

These values may represent overestimations in that any tendency to armor and stabilize is not accounted for. On the other hand, the implicit assumption of perfect phasing tends to underestimate the baseline drop, and thus the degradation and sediment yields.

The aggradation rate for Reach 2 for the first year of project conditions can be computed from Equation 20 and these new gravel infeeds. The calculated value of Δz is +4.0 mm/year, or about 46 times the value obtained by ignoring tributary degradation. However, this represents only an average aggradation rate over the entire reach. Initially this aggradation should take the form of large, rapidly-building bars localized downstream of the mouths of the tributaries. It is these bars that have the most potential to capture fines, and thus become unsuitable for spawning. Downstream of these bars, the aggradation should be gradual, so that fines can be washed out; thus most of Reach 2 would likely remain suitable for spawning.

It should be noted that the tributary degradation calculations have been performed under the unverified assumption that no controls such as bedrock outcrops exist in the reaches under question.

LONG TERM CHANGES IN CHANNEL GEOMETRY

The long term modification of Poplar Creek as it attempts to find a new equilibrium can be surmised in terms of a stable Reach 1 and tributary degradation that has run its course.

Since flood discharges have been lowered in Reaches 1 and 2, it makes sense to assume that the channel will respond by narrowing and losing capacity via vegetation growth and collection of fine material from local sources. This reduction in width can be estimated from Equation 1a if some kind of "surrogate" bankfull discharge for project conditions can be estimated. I did this by calculating discharges which under project conditions have the same exceedance probability as bankfull discharge into a true bankfull discharge. Before this happens, width at the "surrogate" bankfull discharge can be estimated from Equation 14, yielding values of 90 m for Reach 1 and 112 m for Reach 2.

After the adjustment takes place, the eventual bankfull width can be estimated from Equation 1a cast in the form

$$\frac{B_{bf}}{B_{bi}} = \left(\frac{Q_{bf}}{Q_{bi}}\right)^{0.441}$$

where i denotes initial (preproject) and f denotes final values. The values so predicted are 79 m for Reach 1 and 102 m for Reach 2. Thus, a noticeable contraction in bankfull width is predicted. A contraction of this order will likely occur in Reach 2 eventually, as it will still be morphologically active in the long term. A contraction is also likely to occur in Reach 1 due to debris deposits and vegetation encroachment. Whether it can be predicted accurately from Equation 1a, however, is problematic in that asymptotically the reach should be rendered morphologically dead. Indeed, if width maintenance is heavily dependent on the availability of fines for deposition, channel widening can actually occur downstream of a dam (Einstein 1972). However, this is not likely in the case of Poplar Creek, with its gravelly banks.

Reach 1 should reach stability very slowly by a combination of coarsening and degradation; channel narrowing should abet this process somewhat. Parker (e.g., 1980a) has observed degradation and transition from pavement to stable armor in the laboratory; preliminary results suggest that final armor median size mignt be about 1.3 times the natural pavement D_{p50}, or about 65 mm. Assuming that armor D_{90} is twice the armor D_{50} and roughness height k is twice the armor D_{90}, and employing the Neill criterion for stability, Equation 3, and an assumed value of bankfull width of 79 m, a final stable slope of 0.00153 can be calculated for Reach 1.

In treating Reach 2, I assumed that eventually tributary degradation would run its coarse, and annual tributary gravel yields would return to their natural values in Table 7. With no gravel contributed from the stable Reach 1, then, Reach 2 must convey 168.1 metric tons per year through a channel with an estimated bankfull width of 102 m. A routing using the project flow histogram indicates that channel slope must be equal to 0.00189 for this to happen, assuming no change in gravel composition.

The implication here is that the short term aggradation of Reach 2 will be followed by a slow, long term degradation driven by a return of tributary yields to their former values and channel narrowing. The final channel of Reach 2 will still be active and fit for spawning. Unless corrective measures are taken to clear debris from Reach 1 by, for example, the release of appropriately high flows from the dam, it may eventually become unfit for spawning.

MEANDERING AND THE POOL-AND-RIFFLE STRUCTURE

Kinoshita (1957) has classified free meander patterns according to their sinuousity. Low-sinuosity channels usually have two pools, bars, and riffles per bend wavelength; that is, meander bend wavelength corresponds with the wavelength of the pool-and-riffle pattern. Tortuous channels may have as many as six pools per bend.

Typical meandering gravel-bed streams, including Poplar Creek, are of low sinuosity. Where the meander pattern is free, then, coincidence of the meander pattern and the pool-and-riffle pattern may be assumed.

In much of Reach 1, Poplar Creek impinges on a steep cliff, and cannot be said to be completely free. However, the upstream half of Reach 2 shows a well defined succession of five alternate bars about which the channel displays meandering of low sinuosity. Typical meandering gravel-bed streams, including Poplar Creek, are of low sinuosity. I determined an average linear meander wavelength of 1410 m in this portion.

I compared this value with the predictions from two formulas, the Anderson formula and the second form of the modified Anderson formula (Parker and Anderson, 1976). They take the forms

$$\lambda_A = 72F^{\frac{1}{2}}\sqrt{BH} \tag{21a}$$

$$\lambda_{MA} = \frac{1}{a}\,2\sqrt{\pi}\,F\,S^{-\frac{1}{2}}\,\sqrt{BH} \tag{21b}$$

respectively; lambda denotes wavelength, and $a = 0.707$ in the modified Anderson formula based on data. Both of these formulas appear to work well for both laboratory alternate bars and field bars and bends of low sinuosity. This is illustrated for the modified Anderson formula in Figure 8. I employed Equation 21 in conjunction with bankfull parameters. Predicted wavelengths under natural conditions are summarized in Table 8; λ_{OBS} denotes the observed value.

Table 8. Predicted wavelengths under natural conditions.

	REACH 1	REACH 2
λ_{OBS} (m)	-	1410
λ_A (m)	758	844
λ_{MA} (m)	884	977

The values predicted from either Equation 21a or 21b are on the small side in the case of Reach 2, but not reasonably so. Equation 21b is a little better, predicting a value that is 31 percent lower than the observed value.

I then used Equation 21 to predict meander wavelength to which the stream will tend for short term project conditions immediately after the dam is put in operation, using the "surrogate" bankfull discharges defined previously and the assumption that channel contraction had not yet occurred. In addition, a calculation was performed for the long

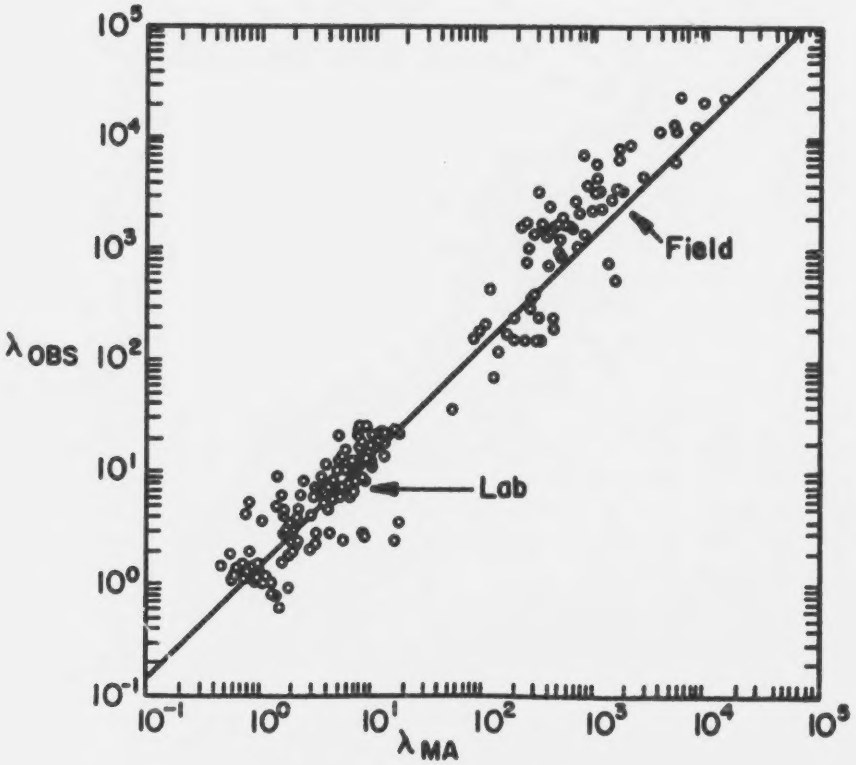

Figure 8. Relation for meander wavelength.

term channel geometry in Reach 2 after contraction has occurred. This was not done for Reach 1, as a long term state of zero mobility is incapable of determining its own meander pattern. Percent changes about the predicted values under natural conditions are also shown below in Table 9.

Table 9. Percent changes about the predicted values under normal natural conditions.

	λ_A	Percent Reduction	λ_{MA}	Percent Reduction
Reach 1 short-term	637	16	726	18
Reach 2 short-term	739	12	843	14
Reach 2 long-term	729	14	830	15

The results indicate that both Reaches 1 and 2 will initially begin to reduce their meander lengths. The process is unlikely to be very effective in the case of Reach 1 due to the near-vanishing transport rate and the control exerted by the cliffs on the south bank. Reach 2 should be able to maintain, in its final state, a meander pattern that is perhaps fifteen percent shorter in wavelength than that observed presently.

The adjustment in Reach 2 should take at least five to ten years, and would likely be disrupted at the upstream end by bars building out from the tributaries. However, the amount of adjustment is not large enough to suggest massive instabilities throughout the entire reach during adjustment.

SUMMARY OF PREDICTIONS FOR "POPLAR" CREEK AND ITS TRIBUTARIES.

The reach of Poplar Creek from the dam to just upstream of Dry Creek (Reach 1) will be almost immediately rendered incapable of moving much gravel by the project. It has negligible water and gravel infeed from tributaries. The gravel that moves is likely to be on the fine side. The low transport rates of gravel (gradually declining from about one tenth of the natural rate) imply extremely slow degradation, with a gradual coarsening of the pavement. After a very long time the bed slope might decrease by a maximum of fifteen percent, accompanied by perhaps a thirty percent coarsening of the surficial material. The maximum potential for degradation is roughly two meters for the reach, but it may take hundreds of years to realize this, and it may never be realized at all if the width in the reach does not contract.

A conventional formula for hydraulic geometry of active gravel streams suggests that if the reach is able to contract and form a new, smaller bankfull channel, bankfull width would be reduced by roughly twenty percent. Some reduction in width can be expected even for this essentially inactive channel (after control) due to vegetation encroachment.

The near-vanishing transport rates also suggest that the substrate will be locked into place. It will suffer only very minor modification due to bedload transport, as most of the coarsening will occur on the surface. However, locally derived fine sand, silt, clay, and organic debris will likely collect in the substrate in increasing quantities under the proposed project flow duration curve, as no mechanism would exist to clean it.

Some cementing of the substrate is also possible. These effects could be avoided or mitigated by modifying the dam operation to allow for occasional short, large releases.

In any event, these processes which act to degrade the substrate of Reach 1 are slow, so that if it is presently suitable for spawning, it should remain so for at least the first few years after commencement of dam operation.

The reach of Poplar Creek from just upstream of Dry Creek to a point upstream of the confluence with the South Fork (Reach 2) presents a different situation. Lowered project discharges should roughly have the capacity of this reach to move gravel. The implied lowered baseline due to flow control should also induce degradation in Dry and Little Dry Creeks, with consequent rapid deltaic deposition at the upstream end of the reach, and initial slower aggradation of the reach as a whole.

Computed gravel infeed rates from the tributaries as they degrade for the first year or two are on the order of 25 times their natural feed rates. The resulting local bars at the upstream end of the reach may thus initially incorporate considerable sand (if it is available) and have substrates unsuitable for spawning. As the aggradation slowly spreads over the entire reach, it is likely that excess fines would be washed out. Most of the reach should maintain a substrate suitble for spawning during this process.

On the order of 10 to 15 cm of degradation should work its way up the tributaries. The amount will be more if tributaries and main stem are far out of phase under project conditions. The tributaries may respond by initially coarsening their bed surfaces; if they are used for spawning this may have a deleterious effect.

In the long term tributary degradation will eventually run its course. Tributary bed structure, gravel load, and gravel content are likely to gradually return to values typical of preproject conditions.

Thus, in the long term the main stem in Reach 2, which was initially oversteepened by transient high tributary gravel inputs, should slowly degrade back to a situation where it can transport near-natural (previous to the project) gravel inputs from the tributaries, with no input of gravel from the main stem upstream.

At the same time as this degradation occurs, the main stem channel should narrow to adjust for the reduced flood flows. I estimate this narrowing to be in the neighborhood of 15%. I also estimate that after this reach goes through its cycle of short term aggradation and long term degradation, it will reach a graded slope very nearly equal to the present value.

Both short term and long term tendencies indicate a reduction in meander wavelength by about 15%. This would probably be accomplished slowly, without an excess of disruption of the channel except at the upstream end of Reach 2 in the deltaic deposits. Here, severe local channel instability may occur.

In brief, Reach 1 should be healthy for spawning in the short term, but should eventually become morphologically dead, with deleterious effects on spawning. Reach 2 should undergo rather severe aggradation, with possible incorporation of fines, at the upstream end in the short term. The rest of the reach should not be too unstable in the short term. The long term prognosis is for a narrower, smaller, but still active Reach 2, that is otherwise not much different from natural conditions, and in that sense, morphologically and biologically healthy.

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EFFECT OF A DAM ON THE MORPHOLOGY OF POPLAR CREEK

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INTRODUCTION

Poplar Creek rises in the coastal mountains, flows eastward and joins the main valley river. A dam is proposed on the main stem of the creek about 17 miles from the mouth. The south fork of the creek enters the main stem approximately 10 miles downstream of the dam site. Two tributaries join the main stream between the dam site and the south fork. One of these, Dry Creek, enters from the north approximately 4 miles from the dam and the other, Little Dry Creek, enters from the south approximately 5 miles from the dam site.

The present paper discusses the effect of the proposed dam on the morphology of the reach of Poplar Creek between the dam site and the confluence with the south fork of Poplar Creek.

GAGING STATIONS

Table 1 lists the gaging stations at which data pertinent to the problem have been collected. The sediment yield for Station 1, which is near the dam site is not given.

In the text that follows, the gaging stations will be referred to by number as follows.

Station 1. Poplar Creek near dam site (No. 11-575810).
Station 2. Poplar Creek near mouth (No. 11-376000).
Station 3. South Fork Poplar Creek near Olinda (No. 11-375870).
Station 4. South Fork Poplar Creek near Poplar (No. 11-375820).
Station 5. Middle Fork Poplar Creek (No. 11-3744).

SIZE DISTRIBUTION OF SEDIMENTS

Data on size distribution of suspended load for several discharges is available for gaging stations 1, 2, and 3. These data are shown in Table 2. The suspended sediment near the dam site (Table 2) is from a few percent to in excess of 50% sand. Small amounts of particles as coarse as 1 mm are found in suspension. The percent of sand in suspension at Stations 2 and 3 are about the same as at Station 1.

As shown by Table 3 the bed load varies from medium sand to fine gravel and tends to be finer than the bed sediment (Table 4). Table 4 gives size distribution of samples taken from the surface of the bed and from samples of material in the upper one foot of the bed. The latter samples are seen to be very coarse and contain particles which will tend

Table 1. Gaging stations in Poplar Creek.

Assigned No.	Station No.	Elev. ft	Watershed area Sq. mi.	Mean discharge cfs/mi^2	Mean Sed. yield T/mi^2 yr	Period of record	
						water	sed.
	1. Main Stem, Poplar Creek						
1	11-575810*	515	395	1.14		7 yrs.	2 yrs.
2	11-376000**	364	927	0.902	853	37 yrs.	11 yrs.
	2. South Fork, Poplar Creek						
3	11-375870	540	371			21 mos.	21 mos.
4	11-375820	525	217	0.982		15 yrs.	
	3. Middle Fork, Poplar Creek						
5	11-3744		249	0.915	1040		

* dam site
**4.7 mi. from mouth

Table 2. Size distribution of suspended load.

Date	Discharge ft^3/sec	Susp. Sed. disch. T/da	Percent finer size, mm			
			0.062	0.125	0.25	0.5)
Station 1.	Poplar Ceek near Dam Site					
3/01/77	135	31				
17/03/77	239	119				
16/03/77	375	1,150	99	99	100	
16/03/77	380	313	99	99	100	
16/03/77	410	982	100			
23/11/77	410	452	99	100		
16/03/77	454	391	99	99	100	
16/03/77	478	1,550	100			
7/04/78	1,600	497	52	56	70	93
14/12/77	2,810	14,900	56	90	93	96
8/02/78	3,180	4,970	47	54	66	90
9/03/78	5,050	12,200	58	69	82	94
19/01/78	5,280	15,400	54	65	81	96
14/01/78	6,030	22,800	69	79	87	96
9/01/78	7,570	56,000	76	84	92	97
16/01/78	12,900	67,900	98	100		
Station 3.	South Fork Poplar Creek					
2/02/77	12					
1/06/77	22	5	94			
8/09/78	26	61	100			
1/12/77	52	4	98	100		
11/05/77	56	12	90			
22/03/77	60	17				
16/03/77	101	62	99	100		
3/01/77	109	111	99	100		
17/03/77	189	389	100			
20/12/77	293	126	100			
23/11/77	352	1,740	100			
16/03/77	372	3,430	96	99	100	
7/04/78	710	196	86	90	94	100
14/12/77	753	5,920	97	98	99	100
3/03/78	980	606	68	78	91	100

Table 2. (Concluded)

Date	Discharge ft^3/sec	Susp. Sed. disch. T/da	Percent finer size, mm			
			0.062	0.125	0.25	0.5)
Station 3. South Fork Poplar Creek (continued)						
23/12/77	1,860	15,900	92	96	98	100
15/12/77	2,020	18,400	96	98	99	100
10/01/78	2,300	10,900	78	87	94	99
9/03/78	2,320	4,770	67	77	90	99
9/01/78	8,830	201,000	77	88	98	100
14/01/78	9,180	25,200	74	94	100	
Station 2. Poplar Creek near Mouth						
7/09/76	62	2	76			
3/03/77	74	3	83			
1/06/77	77	8	85			
21/09/76	95	33	99	99	100	
1/06/77	101	8	92			
22/03/77	152	18	99	100		
8/04/77	154	18	99	100		
20/09/76	293	709	100			
3/01/77	330	225	99	100		
12/05/77	339	62	95			
28/11/77	359	42	98	100		
17/03/77	627	1,780	99	100		
20/12/77	850	207	96	98	100	
23/01/78	2,980	2,060	55	67	83	97
6/03/78	4,920	10,700	54	65	85	95
15/12/77	5,460	50,300	92	93	95	97
19/01/78	6,460	21,600	59	74	90	98
10/03/78	6,780	12,700	47	58	78	95
8/02/78	7,400	18,000	54	65	85	95
8/03/78	17,400	99,600	50	66	84	95
8/03/78	19,100	122,000	58	75	90	96

Table 3. Size distribution of bed load.

Date	Disch. ft^3/sec	Bedload disch. T/da	Sediment Size (mm)				
			d_3	d_{16}	d_{50}	d_{84}	d_{95}
Station 1. Poplar Creek near Dam							
14/13/77	2800	534		0.5	4.3	14.8	18.5
17/03/77	247	1		0.74	1.3	2.0	2.8
Station 2. Poplar Creek near Mouth							
17/03/77	607	8	0.15	0.22	0.40	0.72	1.13
22/03/77	152	3	0.18	0.33	0.62	1.30	4.0
23/01/78	2920	87	0.18	0.34	0.90	3.65	8.0
Station 3. South Fork Poplar Creek							
16/03/77	151		0.20	0.086	0.32	1.13	2.0
17/03/77	198	1	0.25	0.61	1.80	8.0	13.5
6/06/78	204	4		0.56	1.88	4.4	7.3
23/11/77	338	3	0.20	0.46	1.65	14.0	
16/03/77	461	4	0.092	0.32	0.82	4.0	8.0
2/03/78	524	45		0.32	0.45	1.15	3.4
26/01/78	561	323	0.56	1.07	3.15	6.2	9.5
8/02/78	2150	134	0.19	0.68	6.10	16.0	

Table 4. Size distribution of bed sediment.

Sample No.	Distance to dam ft	Size, mm					Notes
		d_3	d_{16}	d_{50}	d_{84}	d_{95}	
Station 1. Poplar Creek near Dam Site							
	1,300	0.36	1.13	4.8	17.7	36	Q = 3670 cfs[a]
	1,300	0.29	0.44	0.88	3.1	8.6	Q = 1520 cfs[b]
Poplar Creek, Dam Site to South Fork							
2-F-78-1	500	0.2	0.34	1.5	9.6	28	Dutch Gulch
2-F-78-6	1,000	0.1	2.0	41	130		
2-F-78-10	29,000	0.2	2.0	30	79	94	Little Dry Creek
2-F-78-12	30,000	0.5	9.4	75	120	150	
2-F-78-14	52,000	0.2	1.2	18	43	62	Near S. Fork
Tributaries to Poplar Creek							
71097	23,000	0.6	1.9	20	59	80	Dry Creek[b]
71101	29,000	0.27	0.88	10	58	100	Little Dry Creek[b]
2-F-78-16	52,000	0.1	0.78	8	27	40	South Fork
Station 2. Poplar Creek near Mouth							
		0.29	0.78	5.3	17.6	20	Q = 7310[a]
		0.19	0.72	6.2	17.3	20	Q = 2960[a]
		0.21	0.52	1.9	12.0	19	Q = 17600[a]
Station 3. South Fork Poplar Creek near Poplar							
		0.27	0.50	4.3	14.8	18.5	Q = 2180[a]
		0.29	0.88	9.6	33	35	Q = 2270[a]

[a] Surface samples

[b] Samples from top 1 ft. of bed

to armor the bed. The 3% size (the size for which 3% of the sediment is finer) of the bed sediment at Station 1 is approximately 0.3 mm. Material finer than this size can be considered to be wash load. From this, one can judge that sediment in this size range is carried easily by the flows without appreciable deposition on the bed (material coarser than the wash load is bed sediment load). The 3% size of bed sediment of Poplar Creek near the mouth (Station 2) is from 0.2 mm to 0.3 mm indicating that sand in this size range is carried through the river system without appreciable deposition.

DISCHARGE OF BED SEDIMENT NEAR DAM SITE (STATION 1)

The sediment yield for Station 1 is not given. However, it can be estimated from measurements at Stations 2 and 3. The sediment yield for Stations 2 and 3 are respectively 853 T per square mile and 1040 T per square mile. It will be assumed that the sediment yield for Station 1 is 1000 T/Mi^2. Data for 1977-78 (Water Resources Data, 1977-78) at Station 1 indicate that the bed load discharge was about 6 percent of the total yield. Based on this percentage the bed load yield is estimated to be 60 T/mi^2 and the suspended load yield is 940 T/mi^2.

To estimate the bed sediment yield we first assume that sediment finer than 0.25 mm is wash load. Data from Station 1 in Table 2 indicates that in 1977-78 no sediment of this size was in suspension until the discharge exceeded 1000 ft^3/sec. For the higher discharges the suspended load contained from 7% to 34% of sediment coarser than 0.25 mm. Based on these percentages the bed sediment yield in the suspended sediment is from 66 T/mi^2 to 320 T/mi^2. The latter yield seems excessive.

The total bed sediment yield is taken as the sum of the yield of suspended sand and the total bed load yield. Assuming the above quantities the total bed sediment yield is from 126 T/mi^2 per year to 380 T/mi^2 per year.

Once the dam is built it is assumed that all the suspended load coarser than 0.25 mm and all the bed load will be deposited in the reservoir. The water discharged from the reservoir will be free of bed sediment and will tend to degrade the channel downstream from the dam.

CHANNEL ARMORING

To investigate armoring, estimates of the bed shear stress for flows in Poplar Creek with several return periods are made. To do this, river slopes will be assumed to lie between .0014 and .0021 (Figure 1) and a Manning friction factor of 0.03 also will be assumed.

These calculations were made for four cross sections, two near Dry Creek and Little Dry Creek (Sections 70 and 71) (Figure 1) and two 1400 ft upstream of the confluence with the South Fork of Poplar Creek (Sections 57 and 59). The dimensionless shear stress for stable particles was taken as 0.047. The results of these calculations are given in the following Table 5.

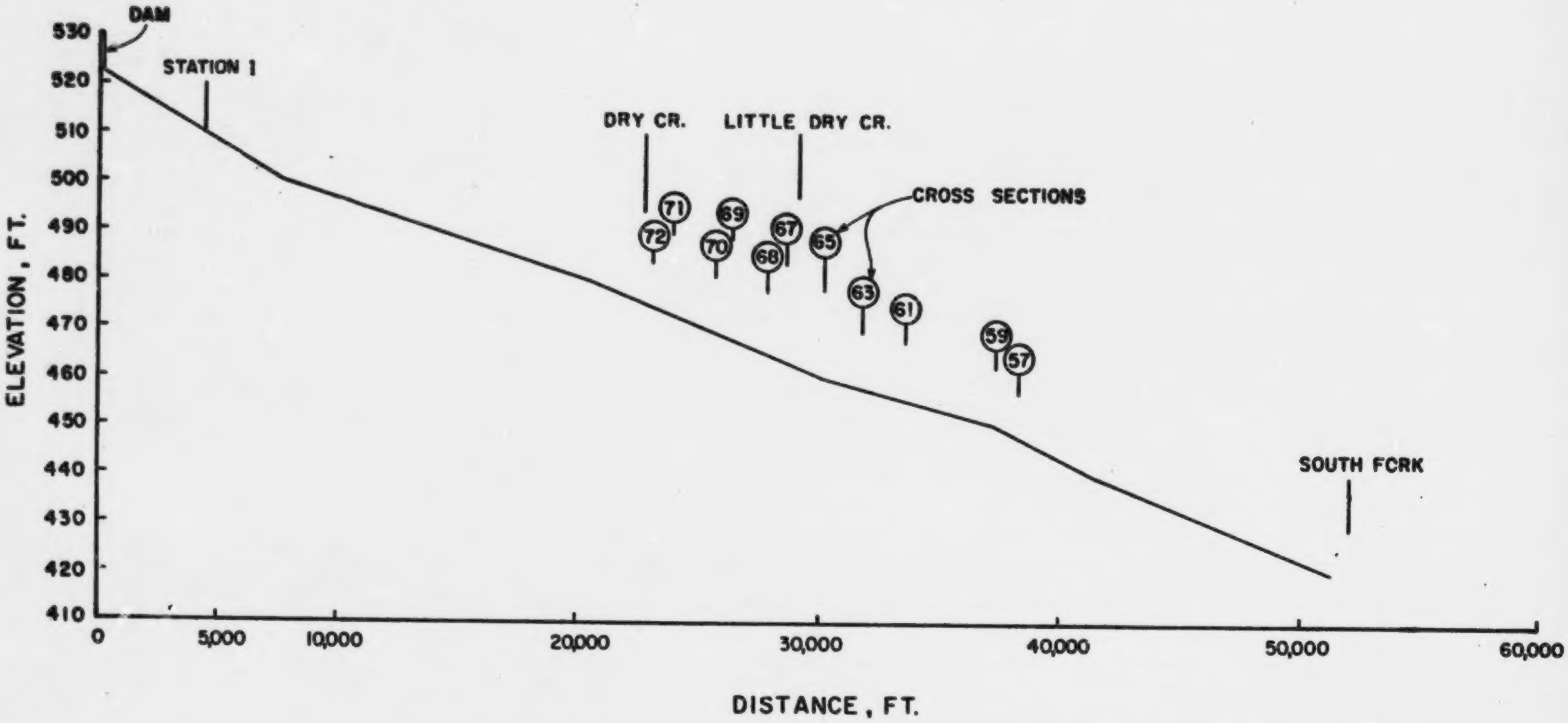

Figure 1. Profile of Poplar Creek.

Table 5. Stable Particle Size for Flows of Various Return Periods.

Return Period	Discharge ft^3/sec	Range in Size of Stable Particles, mm Sec. 57 & 59	Sec. 70 & 71
100	4000	28-50	26-47
50	2400	24-40	21-35
20	1200	18-27	15-26

The flow rates in this table are releases from the dam for the return periods indicated. The flows at the four sections for the return periods indicated would be greater than those shown by the amount of the local runoff. However, the bed sediment downstream of the dam has ample quantities of very coarse particles which would be stable in flows considerably in excess of those listed in Table 4. For example: for a flow of 6000 cu ft per sec a 52 mm particle would be stable at Section 71 and a 59 mm particle would be stable at Section 47. As shown in Table 4 the bed of the creek between the dam and the South Fork contains particles in the 95% size that are as large as 150 mm.

Based on these rough calculations and the information on bed sediment size, it is concluded that the bed of Poplar Creek will armor after the dam is built. The bed will tend to degrade somewhat but degradation will be checked by armoring.

EFFECT OF TRIBUTARIES TO POPLAR CREEK

Dry Creek and Little Dry Creek with drainage areas of approximately 45 sq mi and 40 sq mi, respectively, are the main tributaries to the 10 mile reach of Poplar Creek between the dam site and the confluence of the South Fork. These tributaries will bring in sediment to the main stem as will other minor inflows. The total watershed area of the 10 mile reach of Poplar Creek immediately downstream of the dam, including that of the two main tributaries, is approximately 100 sq mi. Sediment contributed to the Creek from this watershed area will tend to aggrade the stream channel if the flows are incapable of transporting it.

The mean annual sediment yield of the watershed of the 10 mile reach of river downstream of the dam is estimated to be approximately 1000 tons per sq mi or approximately 100,000 tons. The sediment yield at Station 1, with a watershed area of 395 mi^2, is roughly 395,000 tons per year or almost 4 times the sediment yield of the watershed draining to the 10 mile reach of river downstream of the dam.

The proposed dam will modify the distribution of flows from what they are under natural conditions. Because the dam will regulate the flows to the creek the capacity of these flows to transport sediment will be less than that of the unregulated flows.

The mean annual sediment transporting capacity of the flows released from the dam was calculated assuming that the suspended sediment transport relation for the creek was the same as for Station 1, under natural conditions. The flow distribution was furnished. The suspended sediment transport curve was obtained by fitting a curve by eye to data for Station 1 from March 1977 to June 1978. For water discharges less than 2000 cu ft per sec, this relation is (Figure 2)

$$G_{ss} = 63 \times 10^{-6} Q^{2.36} \qquad (1)$$

in which G_{ss} = suspended sediment discharge in tons per day and Q = water discharge in cu ft per sec. Applying the relation shown in Figure 2 to the distribution of releases from the dam gave a suspended sediment yield of 1027 T/da or 375,000 T/yr. Adding 6% to this for the bed load yield gives 397,000 T/yr as the estimated sediment transporting capacity of the flows released from the dam. This is almost four times the contribution of sediment from the dam to the South Fork. The actual transporting capacity should exceed the above amount because the flows will exceed those released by the amount of the local runoff.

Based on the above analysis it is concluded that the reach of Poplar Creek from the dam site to the South Fork will not tend to aggrade. On the contrary the sediment entering the creek downstream from the dam will be carried through the reach and some degradation will take place. As indicated in Section 5 of this report the degradation will be limited by armoring of the bed.

CHANNEL CHANGES

From the analysis above it appears that the flows downstream from the dam have more than enough capacity to move all the sediment reaching the stream in the reach of interest to this study. This means that the stream will tend to degrade the channel. Because the bed sediment is coarse the bed will armor after very little degradation has occurred. As degradation is taking place and especially after it has occurred the stream will erode its banks. This will tend to increase the width of the meanders and the sinuosity of the stream.

'The changes in the stream are not expected to change appreciably the system of pools and riffles.

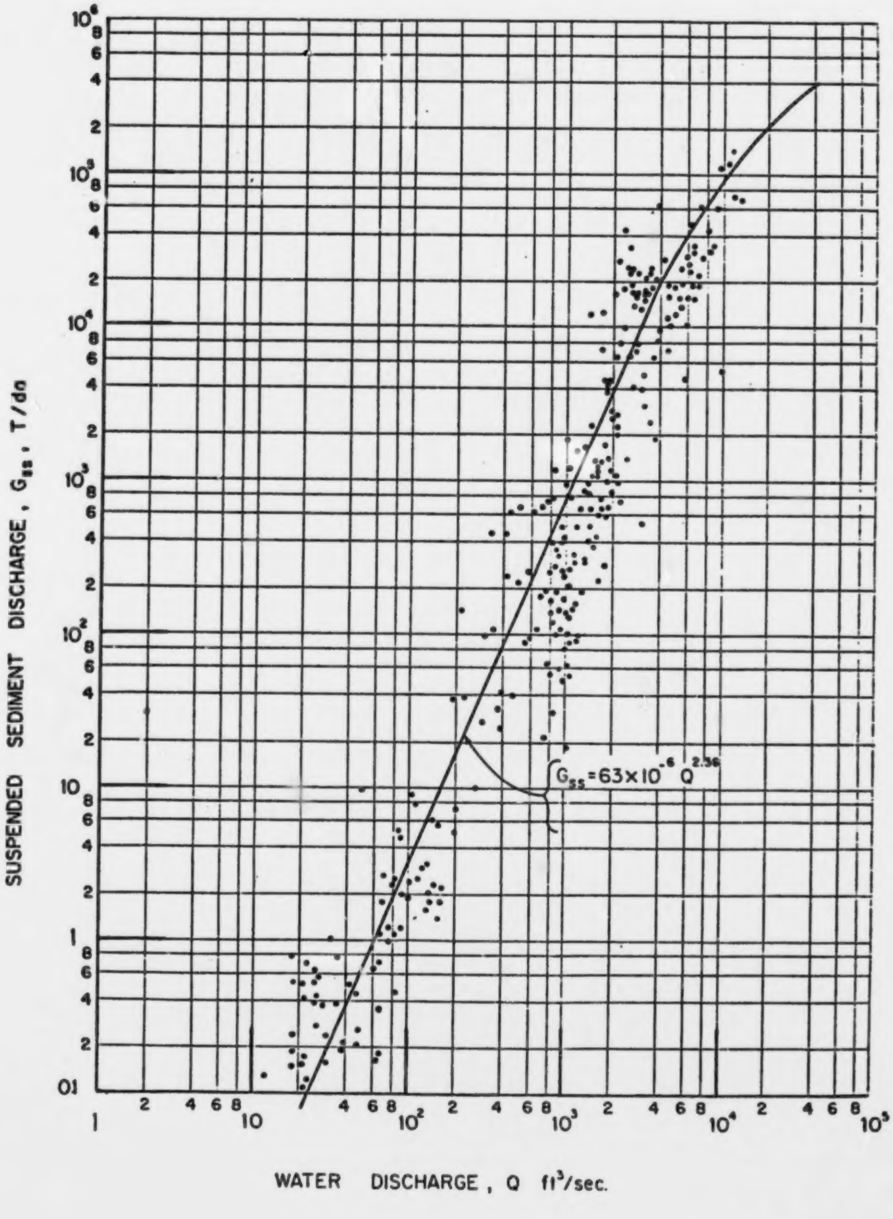

Figure 2. Suspended sediment transport curves for Station 1 near dam site.

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ASSESSMENT OF CHANNEL MORPHOLOGY
ON POPLAR CREEK, CALIFORNIA

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PROBLEM

A set of data has been given to several engineers with knowledge of alluvial rivers. Each was asked to examine only the furnished data and not gather any additional information on the project nor to visit the site. Specifically we were asked: "What will happen to the morphology of the stream's channel as a result of the changes in streamflows and sediment discharge caused by the construction of a reservoir upstream of a reach of stream?" Specifically, for both a short and a long time after construction of the reservoir:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrata material? and
4. What will be the pool-riffle sequence?

METHOD OF ANALYSIS

In the final analysis of this type of engineering problem there are two methods that can be used.

1. A quantitative analysis based on hydraulic data.
2. A mathematical model of channel and watershed process.

Regardless of the quantity and type of data available, before either of the above techniques can be effective in determining present conditions and in predicting future trends, a geomorphic analysis is an absolute requirement. Even if there are only minor hydraulic data available for the quantitative or the math model approach, the geomorphic assessment will allow engineers, economists and environmentalists to make a good qualitative analysis of the drainage basin.

The initial base of a geomorphic study should be a thorough on-site examination of the drainage basin, followed by a research of available climatic, hydrologic, hydraulic, and geologic data. Next, compile a complete history of man's activities in the basin. This should include both land and water uses as well as all engineering projects that would affect the movement of the water and of the sediments. No matter what future engineering analysis is anticipated, this initial geomorphic look is of prime importance, and without it any future engineering interpretation is at best only temporarily factual.

A full scale text book would be necessary to present the geomorphic approach, but the author thought it to be appropriate to include a condensed model of the relationship of all the variables operating on and within the drainage basin.

GEOMORPHIC MODEL

Geomorphology is the science of land forms, the Earth's shape resulting from the forces acting on the landforms and their development through time. Fluvial geomorphology is the study of the action of water in changing the Earth's landforms as well as the water's influence on man's use of the land.

Schumm (1977) states that, "Development of an understanding of any part of a river system depends to a large extent on the appreciation of both upstream and downstream controls; therefore, the fluvial system must be considered as a whole ... an attempt (must be made) to consider the fluvial system and its components in such a way that the interaction of the components and the resulting degree of inherent instability of the system can be comprehended and related to some concerns of the economic geologist, geomorphologist, stratigraphor, land manager, conservationist, and civil engineer." Allen (1970) indicates that a drainage basin with its arrangement of streams is a dynamic system in which the intrinsic as well as the extrinsic variables express their behavioral changes both in time and in space. Additionally, the general shape of a drainage basin and the physical properties of the sediment are a function not only of space and time but also of the basin's geologic history. All the variables that influence the discharge and the sediment load of a drainage basin are the result of the basin's geologic history. This includes the type and depth of suballuvial formatons, the gradation of bed and bank material, the valley slope, and the hydrologic regime. These combine to produce the hydraulic characteristics of the basins and rivers. Lowdermilk (1953) shows how, over the past 7,000 years, man has had a strong influence on the Earth's drainage basin characteristics, and how man has in many basins grossly mismanaged the use of the land and the water.

One of the most difficult tasks for political, engineering, and environmental groups to understand is that nature is constantly changing. Some of the changes are catastrophic in the form of an earthquake, volcano, flood, or forest fire. Others are slow and almost unnoticeable in engineering time, such as the forces of water in changing the shape of the land and the characteristics of rivers. The term "Dynamic Equilibrium" is often used. This does not mean that all forces and reactions are cycling around some norm, but that whenever there is a catastrophic or a slow change, nature will respond and return to an equilibrium condition. This response can be fast but is usually slow and the new equilibrium state is often not the same as it was originally. We as engineers are conditioned to think in engineering time frames and to consider the data we collect, in designing any project, as being static or cyclic around an average or frequency of return of an event that will always follow the same timely pattern. This concept can be and usually is wrong. As soon as we build any engineering project, we alter nature's mode of operation and the dynamics of nature will respond and produce a new and often very different state of "Dynamic Equilibrium." In particular, variables of the fluvial system are very responsive to man's activities and engineering efforts.

Often nature has problems, because it is responding to some unbalancing event and is in a transition state, attempting to return to a balanced equilibrium. The data we collect are often taken from a transitory unbalanced system and therefore the response of the altered system can be very costly to maintain. Also, we as engineers must recognize that when dealing with the forces of nature we can never design the perfect end product. In river engineering endeavors, only by completely understanding the system, can we hope to construct projects with the least number of problems and with the least costs of maintenance.

Before design or modification of any project on any drainage basin, a geomorphic analysis must be made of that basin. The fluvial system within any drainage basin is extremely complicated and, as the following explanation will exemplify, can be in a fast or a slow response trend in a wide variety of conditions. Determining this condition and the trend of the basin's natural response is necessary in order to predict the response of the drainage system to the engineering efforts. Thornbury (1969) lists 10 concepts relevant to geomorphology and to fluvial systems. Schumm (1977) narrows these 10 concepts to 3. The first is uniformity, which means simply that the laws of Newtonian physics and chemistry controlled the operation of past erosional and depositional processes as they do today; the second principle is that within the constraints of geology (structure) there is a determinable sequence of landscape evolution through time (stage); and the third principle, which is obviously in conflict with the second, is that both landscape history and Earth history are complex. This complexity is related to the external influences of climatic change and the evolutionary alterations of the Earth's surface. Schumm (1973) offers an additional concept which states that geomorphic systems can be strongly influenced by thresholds. That is, abrupt changes may occur during landscape evolution, as threshold values of stress are exceeded. The reader can gain a more complete understanding of these concepts by reading the indicated references.

Schumm (1977) shows that any drainage system at any point in time could be cyclic, steady, progressively changing, or graded. He developed the model shown in Figure 1 of an erosional landscape over varying periods of time and places a possible time frame for each identifiable phase. Schumm states that dynamic, steady-state, and static equilibrium define the type of land-form behavior assumed for cyclic, graded, and steady time spans...but metastable and dynamic metastable equilibria may also be needed to be included because they reflect the influences of thresholds that can cause abrupt episodes of system adjustment. Thresholds result from a gradual increase of external or internal stress, or both, that eventually produces a dramatic response of the system.

Many types of variables have an influence on the characteristics of a drainage basin and their basic interrelationship is extremely complicated. Figure 2 is a qualitative model of these interrelationships.

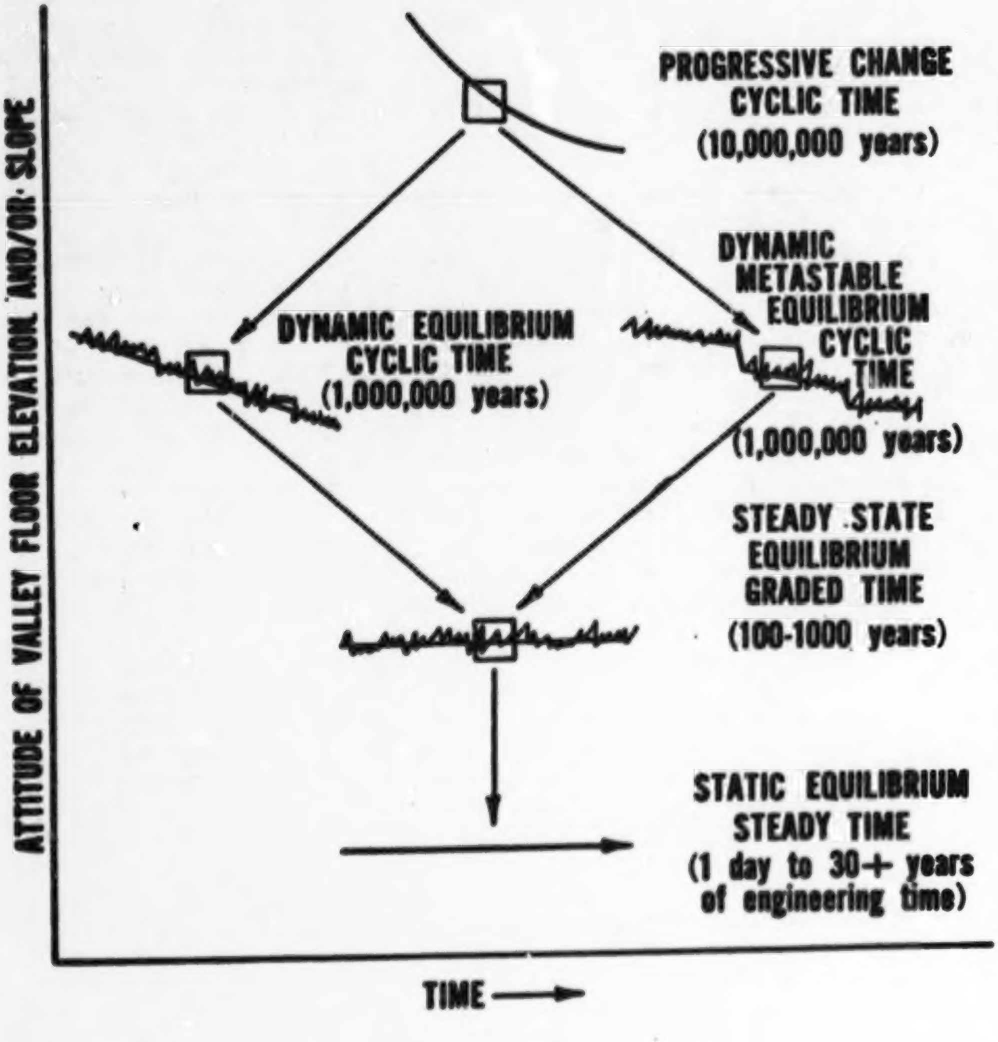

Figure 1. Model of landscape erosional evolution (adapted from Schumm 1977).

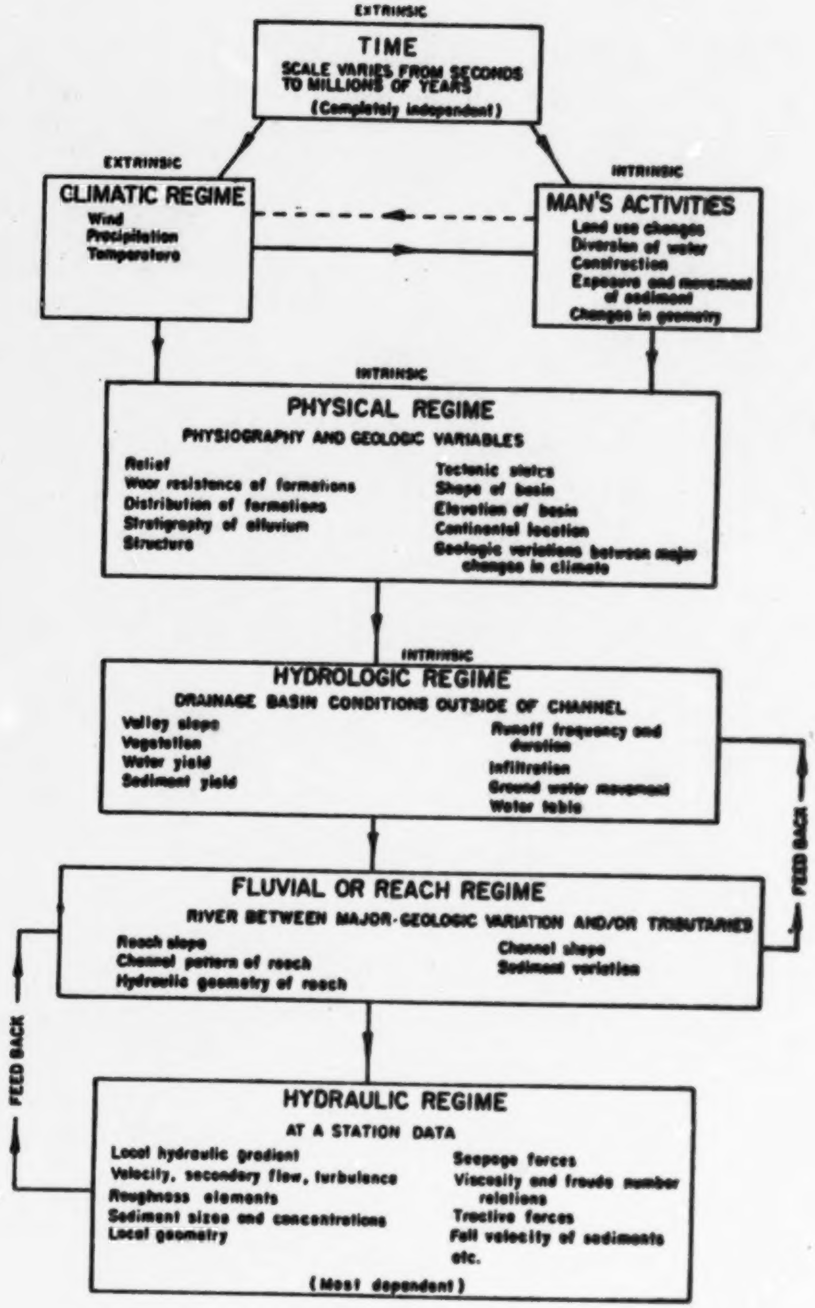

Figure 2. Geomorphic model of a drainage basin.

Time is the most independent of all variables but it does not operate on all parameters on the same scale. It varies from seconds of time in the hydraulic regime to millions of years in the geologic regime.

The Earth's surface is in contact with the atmosphere; this causes erosion. The atmosphere powered by the sun is in constant motion, creating the climatic variable which provides the energy and the mechanisms to shape the geologic variables. Historically, climate would be considered a very independent variable; however, in recent times, man has acquired the ability to modify the weather and possibly to create long-lasting weather alterations accidently. Man's activities are mostly intrinsic and may be either dependent or independent. He operates mostly on the physical regime and alters the normal timetable and quantity of response.

During a basin's geologic history, the climatic variables, operating on the complex physiographic and geologic variables, shape the basin and produce the sediments which determine the nature of today's rivers plus the use of the basin's land. Throughout history, man has altered many local and regional geologic processes. Man creates landforms in two ways: first, as the direct instrument of change, when he wields a spade or a bulldozer; secondly, through his diversionary influences upon other geomorphological processes, as when he channelizes and stabilizes a river and prevents its meandering or alters its flow. His direct influence may be either purposeful or incidental to other activities (Allen 1970).

Climate, and now mankind, determine the character and the behavior of a drainage basin by operating on complex physiographic and geologic parameters. Geologic variables are not readily quantified but are predominantly the lithology, stratigraphy, general geologic structure, and tectonic status of the basin. The process is not steady but highly variable in response to ever-changing climatic input and to variation in erodibility of the basin formations. Man's exposure to any drainage basin is but a moment in time and nature may be in a transitive state, attempting to adjust to some imposed set of conditions. Therefore, nature is not always in the right condition for man's needs and can often be improved upon with a reasonable use of man's intelligence.

The action of climate and man on the physical regime produces the hydrologic regime which determines the rate and the mode of movement of the sediments and water into the basin's streams. The valley slope, i.e., the platform on which the river flows and the quantity and the type of sediment and discharge are the products of the hydrologic regime.

The interaction of the hydrologic parameters produce the fluvial regime. This is the slope and geometry of a stream reach. These interact to produce the hydraulic regime which is determined by the local hydraulic conditions. This hydrologic regime is the current or present integrated response of the entire stream basin history to that moment.

After sediment, within the stream, begins to move, conditions change so that there must be a feedback between the hydraulic regime and the fluvial regime and between the fluvial and the hydrologic regimes. These interactions set up a new group of conditions that change the fluvial and the hydraulic regimes. This is the reason that a complete geologic and historic analysis must be made of any drainage basin before imposing changes in water and sediment conditions. It is necessary to know, at least in a qualitative way, what the trends of response of the controlling physical regime are within the chain of interactions in the hydrologic, then the fluvial, and finally the hydraulic regimes.

Nearly all data collected on any river system are the variables of the hydraulic regime. These are the most dependent of all the many variables of climate, geology, soils, and hydrology of the drainage system. These hydraulic variables only occur at the time and place collected, and are constantly changing in response to the local geometry of the channel, which in turn is the result of the basin's geologic history.

Too often we attempt to analyze problems by only considering local conditions. A drainage basin should be considered a live entity. All of the many variables are integrated so that any change in one portion of the basin can in time be felt throughout the entire basin. However, these reactions and the stream's response are often arrested upstream by a geologic or manmade control and downstream by a hydraulic control. It is necessary that we understand the natural trends and responses of a drainage system so that we can better predict its reaction to any change imposed by man in order to minimize maintenance and insure more successful use of our waterways.

On this project, because of the limited data, the author has chosen to analyze the problem using a generalized geomorphic approach.

DATA AVAILABLE

Following is a list of the limited data furnished to the author.

1. Considerable discharge and sediment data over a brief period were furnished at three stations in the vicinity of the proposed project.

2. Topo maps.

a.	1958	1:250,000	100 and 200 ft contours
b.	1947/1952	1:62,500	25 and 50 ft contours
c.	1964/1965	1:24,000	10 and 20 ft contours

3. USGS Water Supply paper giving generalized information on water and sediment characteristics.

4. Location maps.

5. Limited cross sections of river over a short reach.

ADDITIONAL DATA REQUIRED FOR A MEANINGFUL INITIAL ANALYSIS

1. Aerial photos over as long a period of time as possible.
2. Boring logs to see what substrata looked like.
3. Geologic history of the basin.
4. Data on placer mining activities.
5. Land use data.
6. A visit to site. Anyone attempting to analyze a stream's characteristics or possible response, without physically visiting the site, is asking for criticism.
7. Size of the bed and bank material from the entire drainage basin.
8. U.S. Corps of Engineers (1970) report.
9. Information on response of other streams in the area with similar projects.
10. Complete set of maps instead of limited location maps as furnished.
11. Data on downstream activities that might alter the base level.

This initial analysis should point out any additional data required for complete design and regulation of the project.

INFORMATION GLEANED FROM FURNISHED DATA

Climate Regime

1. Average rainfall varies from 25 inches per year at the lower end of the basin to 70 inches per year at the divide. One storm in January 1974 dumped, in a 4-day period, from 4 inches of rain at the lower end of the basin to 16 inches at the divide.
2. Records indicate an average runoff of 30% but a storm this heavy during a wet period could produce over 90% runoff and move excessive sediments into the channel which subsequently reduced reservoir flows could not convey. This action could initiate aggradation and possible braiding.
3. Wet cool winters with 80% of moisture and dry hot summers. This type weather cycle is usually conducive to high sediment production under abnormal weather cycles.

Man's Activities

1. There has been considerable land clearing or loss of vegetation between the 1947 and the 1965 maps, mostly in the 1958 to 1965 period. What has happened in the past 15 years? What is the future land use to be, agricultural with more irrigated land from the reservoir water, or urban and recreational? Either would induce removal of natural vegetation and probably increase the runoff and sediment production. These changes would further aggravate tendencies to aggrade and braid the channel below the dam.

2. The 1965 maps do not extend above the damsite, but the 1:62500 scale older maps indicate considerable placer mining. Usually this activity would wash out most of the fines in the bed and leave mostly the coarse material. This might be the reason for the relative stability of channel indicated in the 20 year period covered by the furnished maps. This excessive coarse bed material would tend to armor the bed and probably prevent any future degradation below present bed elevations. The tributaries below the dam have had similar mining activity. An onsite inspection would help clarify these questions.

3. Orchards on river bank denote stream stability. These seem to be on the first terrace level and 10 ft above stream. This would suggest that flood flows are infrequent and that they do not carry large amounts of coarse material that would be harmful to agriculture.

4. Maps show irrigation ditches. How much water is removed from stream during the dry summer months and will this be increased with the reservoir? If so, increased groundwater return to the stream could activate caving. The amount of gravel shown on the maps indicates good subsurface water movement and well drained land; therefore, an increase in irrigated water during the dry season would probably increase production of the fine sediments.

Physical Regime

1. The contours and the variation in sediment production between the upper and lower portion of the basin indicate different geologic formation with possible controls that are or could influence channel slopes and shapes.

2. Where the stream is close to a hill line below the dam, it is relatively straight. This indicates a nonerodible bed and bank and a control of the hydraulic geometry of the channel.

3. Map contours indicate a terraced topography and a possible degradation cycle. Reduced flow coupled with raising the downstream base levels could reverse this trend.

4. Terrace contours do not seem to show any signs of stream meanders. This could indicate that terraces are old, formed during a previous geologic period. This could indicate long time stability so reservoir action will probably have little effect on the channel as long as sediment production below the dam is controlled.

5. Terraces and stream banks should show intermittent lenses of sand and gravel which could effect bank stability by armoring the toe and also promote bank instability by groundwater movement leaching out sand lenses causing banks to settle.

6. Maps indicate an abundance of gravel on the bars and in the tributary flood plains. What is the size of gravel and is it only a surface armoring or is the entire bed of the stream composed of gravel to some depth?

7. What is the potential for tectonic activity in the area? Only onsite inspection, boring logs, and a study of the area's geology will answer these questions.

Hydrologic Regime

1. The USGS Water Supply Paper (1978) indicates that the bedload of the several creeks (in the report) varies from 2 to 210 tons per square mile per year and that most of this movement is during the short high stream flows which would be eliminated after the reservoir is constructed. Placer mining tailings seem to be very evident in the tributary drainage basins so this available coarse sediment will end up in the main channel during occasional high runoff from the tributaries, and the reduced flow of the main channel may not be able to move it. This would promote gradual aggradation and a tendency to braid.

2. The USGS report states that the loose soil in the hills erodes rapidly. The reservoir should trap most of this and with the apparent gradient of the stream below the dam, the fine material flushed into this reach should be transported out of the system. This could make the water more turbid and also enhance aggradation tendencies downstream.

3. In 1964, the hills were still vegetated yet the USGS report states that the hill soils are easily eroded and a larger portion of total sediment production comes from the hill area. Does this mean that tree cover is very sparse and that ground cover is virtually nonexistent. If so, chances are that changes in land use may not alter present sediment production.

4. The USGS report states that the lakes of this type release only 5-30% of suspended material that enters from upstream. This would promote a tendency to degrade below the dam; however, excess coarse material would stop degradation, and as

the stream attempts to satisfy its appetite for sediment, it will attack the banks, convey out all fine sediment from tributaries and leave nothing but coarse material in the bed. Also, caving banks would probably armor their toe with the gravel (if it is present) in the bank, thus arresting meandering and returning the stream to stability in the reach above tributaries but promoting alternate periods of stability and then instability in the reach between Dry Creek and South Fork as the toe of banks and bars are intermittently stabilized then aggrade over the protection to a point where banks can again be attacked. This action will cause the stream to braid and aggrade until it establishes a slope that effects the tributaries. Aggradation in the tributaries will slow sediment movement and in some long period of time should again return the entire system to a more stable condition. The time for this depends on many factors that must be analyzed through further geomorphic study.

5. A more complete analysis of rainfall-runoff-area of each drainage system below the dam would give a better handle on the potential sediment production of these areas.

6. The USGS report states an increase of low flows from 5 to 32 cfs, medium flows to remain the same, and a reduction of average high flows from 6300 cfs to 5000 cfs, plus the 100-year frequency to be reduced to 40% of the 5-year flow. This suggests that river below the dam will not be able to move the coarse sediment but should continue to move all sizes of sediment below the more coarse gravel. No data are available on sizes of the coarse materials in the tributaries.

7. The USGS report states that total sediment production above the reservoir is two to four times higher than that below the reservoir. This could minimize the aggradation-braiding tendencies and indicates that the response could be very slow.

Fluvial Regime

1. All tributaries have gradients that are at least four to five times steeper than the main channel; therefore, they probably can initially transport any size sediment available, which will surcharge the main channel with coarse material it cannot move.

2. Cross sections furnished indicate, in the short reach surveyed, that both the top bank and thalweg have the same gradient with a fairly constant bank height. However, the cross section area from these surveys varies from 1800 sq ft to 6700 sq ft with an average of 3600 sq ft. This wide variation in cross section area indicates potential differences in bank stability. These areas could be the location of future meander growth.

3. If the county line on the 1965 maps is a fixed boundary then the stream has not moved in a long time. If the county line

is always at stream center then these criteria are of no analytical use.

4. The stream near the town of Cottonwood is in a braided condition, but very little change is evident (from the maps) upstream of the confluence of South Fork. Is this because of slope change downstream from either sediment production and/or possible change in base level from some other engineering project or downstream aggradation tendencies? Further knowledge of the system is needed.

5. The cross section area of the surveyed section of river shows an average of 3600 sq. ft, with an average high flow of 6300 cfs. This indicates an underfit stream with velocities too slow to transport any but the fine sediments or a degrading situation which would tie in with the several terraces in the valley. But this would conflict with furnished data and indicate channel characteristics. Only field inspection and more data would be needed to resolve this inconsistency.

6. A more complete survey of the channel geometry is needed in order to identify its present hydraulic-geometry. It is also necessary to know the location of the vegetation and whether it is small and temporary or of a permanent nature. With this knowledge, present trends and future conditions could be easier to predict.

7. Several chutes on the inside of the point bars are indicated on the topographic maps. Only further knowledge and site inspection will clarify if these are permanent high water channels or developing cutoffs. There does seem to be some indication of past cutoffs in the present channel. Determining the nature of these will help determine future gradients of the channel.

8. What effect will the change in base level by the reservoir have on the channel upstream?

Hydraulic Regime

1. The only data available are from a few stations. This only tells what is happening at that point in the system and to some degree gives an indication of long time conditions (for the period of the data) of the reach between the proposed dam and the town of Cottonwood some 11 miles downstream.

2. An analysis of these sketchy, inadequate data only indicates the normal cyclic variation with stream flows and does not show any long term changes to the system that could be occurring but not evident in the short period of record.

3. Using just the hydraulic data furnished could give the designers a false security that only minor changes would be expected. At this point, the author does not believe an

analysis of the data is meaningful until after a geomorphic study.

REACTION OF THE STUDY REACH

Analysis of the data furnished indicates the following predictions.

1. The whole system is currently relatively stable but might be starting to respond to conditions of the stream below Cottonwood, California. This response could be of geologic origin or could have begun during the placer mining activities of the past century and further aggravated by recent land use changes and engineering projects in the Sacramento River Valley.

2. The reach of river between the dam site and Dry Creek will start to meander and possibly have some slight aggradation. This action will cease when the caving banks develop a toe protection composed of gravel in the present bank. From that point forward, this reach should be fairly stable in location and in time with a well armored gravel bed. The pool-riffle sequence should remain about as it is presently.

3. The reach of river between Dry Creek and South Fork will start a slow aggradation tendency and will begin to braid. Banks will be attacked until they are stabilized by the gravel in the banks forming a toe protection. This reaction will continue for a fairly long time with alternate periods and short reaches of armoring and aggradation followed by periods of bank caving as the protection is covered by the aggrading channel. When this occurs, scattered sections of bank will be eroded and the process repeated. As the river channel aggrades, this will set a new base level for both the tributaries and for the upstream reach below the dam. The reach between the dam and the aggrading reach has enough gradient that it can transport the river sediment from its banks until the entire system reaches a new stability regime. The tributaries will aggrade as a result of raising their base level and should then transport less and less of the coarser sediments. As this reach aggrades and braids, the pool-riffle sequence and number and depths of pools should vary considerably until stability is achieved. The trend of the whole system should be toward a reduction in time and magnitude of response until stability is finally reached. Present channel cross section and gradients are large enough to support this process for several cycles but eventually could incorporate the entire lowest terrace of the flood plain. If this happens, the return to stability will be much further in the future. It is also possible, with the gradually widened flood plain and gentler slopes from aggradation, for the river to revert back to a meandering stream after a long cyclic period of adjustment. Activities downstream of Cottonwood and in the Sacramento River Valley will probably be the controlling factors.

4. Alternating long periods of wet and dry climate would have an impact on the time and magnitude of these changes.

5. Increase irrigated farming will also affect the stream flow and bank stability.

6. The condition that would promote the largest and fastest changes would be a localized storm cell over the tributary basins with a resulting high runoff while the discharge from the dam is relatively low.

CONCLUSIONS AND RECOMMENDATIONS

Much more data are needed before an adequate analysis and prediction can be offered. Unfortunately, the data as presented to the author are quite often all that is analyzed prior to many of our engineering efforts. Many of the past and present problems are not so evident or obvious because maintenance dollars have "engineered" us out of trouble.

After a study is made as herein suggested, downstream problems might be minimized by periodically releasing large discharges and flushing out the channel.

In the past several decades, we have learned enough about our rivers and drainage basins that no project should be built or altered without a complete fluvial geomorphic analysis.

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REPORTS ON ELK RIVER WORKSHOP PROBLEM

ELK RIVER

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REPORT ON THE ELK RIVER PROBLEM SESSION PAPERS

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INTRODUCTION

The purpose of this report is to summarize the methods of approach and the predictions by the various authors of the Elk River Problem Session group. The group was composed of five authors who are recognized experts, a moderator, and a reporter.

Each of the five authors prepared papers evaluating the impacts that installing several upstream retarding structures would have on 10 miles of a downstream channel reach called the Longton reach. This reach has a total drainage area of over 390 square miles. The retarding structures control nearly 60 percent of the area.

The moderator, C. Thorne prepared a paper, subsequent to the workshop, discussing modeling and data requirements for evaluating downstream impacts. Therefore, this report will concentrate on the five experts' papers and will consider the moderator's paper to be supplemental but useful to this report.

APPROACH BY THE EXPERTS

All five experts approached the problem from the same overall viewpoint, i.e., each used experience supplemented with case histories. However, their individual approaches varied.

Three experts, H. Chang, P. Lagasse, and M. Stevens, used classic regime approaches, although each used a different set of regime equations.

Another, D. Ralston, used a geomorphical approach. He analyzed the geologic, soils, and hydrologic data and then synthesized the results to predict the stream response.

The remaining expert, E. Grissinger, used his experience gained from case studies in other geomorphological areas, together with his training as a soil scientist. He addressed the stream response entirely in terms of bank stability as related to the susceptibility of the bank toe to erosion resulting from induced prolonged low flow attack. The prolonged low flow is due to the controlled releases from the retarding structures.

SUMMARY OF EXPERTS' PREDICTIONS

Each of the experts predicted the short and long term responses of the Longton reach with respect to: 1) meander pattern, 2) configuration of the channel, 3) substrate material, and 4) pool-riffle existence.

The significance of the extensive geologic bed control forced the attention of all to the importance of bank stability and the role of the existing bank vegetation. These two factors, geologic bed control and dense bank vegetation, resulted in very similar conclusions among the experts in spite of minor specific differences.

Table 1, Summary of Responses, shows the individual predictions regarding some basic dependent variables associated with the above four items. A "0" rating means no change, "-" means decrease, "+" means an increase, and "N.C." means no comment. The ratings were extracted from each paper. They were either extracted directly or were interpreted by the reporter to be inferred. "N.C." means the reporter could not reasonably determine the author's prediction for that specific variable.

Table 1. Summary of responses.

			Bed					Channel Shape		
Author	Bank Stability	Bed Material Size	Elev.	Slope	Sinuosity	Meander Wave Length	Riffle-Pool Spacing	Width	Depth	Width Depth Ration
					Short Term					
Change	0	0	0	0	0	0	0	0	0	0
Grissinger[1]	0	0	0	0	0	0	0	0	0	0
Lagasse	+	-	+	+	0	0	0	0	-	0
Ralston	+	+	0	0	0	0	0	0	0	0
Stevens	+	+	0	0	0	0	0	+	0	+
					Long Term					
Change	+	-	+	+	-	N.C.	-	-	-	0
Grissinger[2]	-	-	+	+	0	0	0	+	-	+
Lagasse	+	+	0	0	0	0	0	0	0	0
Stevens	+	+	0	-	-	+	0	-	-	N.C..

[1] Assumes stable banks.

[2] Assumes unstable bank toes and in-site cohesive bank material.

In spite of any differences between the experts for a specific variable, most authors indicated that what changes might occur would be minor in magnitude because of the geologic and vegetative controls. All authors stressed the importance of the bank vegetation on bank stability.

Grissinger presented three possible scenarios: 1) stable banks resulting in no change, 2) unstable bank toes resulting in bank failure primarily dependent upon hydraulic action, and 3) unstable bank toes resulting in large-scale gravity induced bank failure. However, he felt

that he had insufficient data to predict which scenario was most likely to occur. Also, the Elk River stream system is not underlain by paleosols as were the two other case studies he presented for comparison.

SUMMARY CONCLUSION

This reporter concludes, based upon the experts predictions, that the Longton reach will not change appreciably or that any changes will occur very slowly over both the short and long term, except for a thinning and coarsening of the alluvium over the bedrock channel bottom, as long as the bank vegetation is properly maintained.

Future projects should be screened for potential change using similar approaches including site visits supplemented with applicable case studies. An analysis of bank stability with respect to prolonged flows attacking the bank toes should always be considered.

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MODELING AND DATA REQUIREMENTS FOR THE PREDICTION OF DOWNSTREAM RIVER CHANNEL CHANGES FROM DIVERSIONS OR RESERVOIR CONSTRUCTION

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INTRODUCTION

The development of a hydrologic basin and river system for the purposes of flood control and water resource management will almost always result in changes in the flow regime, the rate of transport and quality of sediment and, consequently, the cross-sectional geometry and plan shape of the channels in the system. These physical changes to the channel in turn affect the ecological habitats in the river system. These effects may or may not be beneficial. However, if design and management procedures based on a sound knowledge of natural processes are used, the effects can be optimized and environmentally undesirable and economically expensive repercussions can be avoided. For example, river training work tends to be most successful where a sinuous channel is maintained. Artificially straightened reaches often attempt to regain their sinuous form. This results in the need for almost continual dredging and bank reveting, both of which are extremely costly and environmentally undesirable (Winkley 1982). Experience like this highlights the importance of working with, rather than against, the natural inclinations of the river.

In this respect, we may question whether the present level of knowledge of natural fluvial processes is sufficient for the purpose of predicting downstream channel changes. A second question then arises. Are the data avilable for natural river systems adequate for the latest prediction techniques to be applied outside highly instrumented experimental hydrologic basins? These very fundamental questions were addressed at this meeting.

In this short contribution, the author, as moderator of the Elk River discussion group, wishes to summarize some of the points he made in discussing these questions.

FLUVIAL PROCESSES

Mathematical modeling of fluvial processes and systems has now developed to the point where future development can concentrate on improving and refining existing models rather than developing new models from scratch in each new study (D. B. Simons, personal communication, 1980). When using mathematical models to predict downstream response to the imposition of diversions or reservoirs, it is very important to consider the whole hydrologic, sedimentary, and fluvial system. However, the evaluation of future ecological habitats in the river requires very detailed data on the variations in flow velocity, boundary shear stress and bed material size, gradation and stability, between pools and riffles, and bends and crossovers. These morphological features have repeating lengths of the order of a few times the channel width. To run a catchment wide model to produce this detailed information for the entire river system would require a huge amount of computing time and core storage. This simply would not be feasible at the moment. The best approach may be initially to use a catchment model to predict overall response and to identify both critical and typical channel reaches. Hydraulic models could then be applied to the reaches which are of particular interest to supply the type of detailed data required for habitat evaluation. This approach to catchment modeling has been adopted and developed by scientists at the U.S. Department of Agriculture Sedimentation Laboratory (Alonso et al. 1978 and Borah et al. 1980).

Perhaps the least well understood aspect of channel response is that of streambank erosion. Yet it is essential that streambank stability and erosion be considered because it has an immediate and very damaging impact on the channel, on the flood plain, and on the people who use or inhabit the river valley. This can result in highly adverse social and political repercussions as well as undesirable physical effects.

Bank erosion plays an extremely important role in the fluvial system as an alternative source of sediment (particularly fine-grained wash load) when the sediment input to a reach is reduced significantly by a diversion or reservoir. Bank erosion also brings into operation rapidly operating and highly complex feedback loops between the hydraulic and sedimentary regimes and channel geometry. These feedback loops operate through changes in plan shape, and particularly through changes in sinuousity, which can result in rapid channel slope adjustment (Hey 1978).

Channel degradation, aggradataion, plan shape changes, or dynamic stability in a laterally active channel can all result in bank erosion under certain circumstances. This makes it difficult to predict the extent of bank erosion which may be experienced as a result of any particular hydraulic change. Also, as almost all rivers erode their banks to a greater or lesser degree under entirely natural conditions, it becomes very difficult to separate normal bank erosion from that caused by a newly imposed, artificial change to the fluvial system.

Bank retreat may be the result of a variety of processes of erosion and mechanisms of failure. These processes and mechanisms are related not only to hydraulic conditions but also to the height, slope and stratigraphy of the bank, to the properties of the bank material, and to climatic conditions especially the intensity and amount of precipitation (Thorne 1982; R. H. Kesel, personal communication, 1980). This makes it unlikely that a single property of the bank material can fully represent the susceptibility of a bank to all the various processes and mechanisms of erosion and failure. One factor common to all eroding streambanks does emerge, however. This is the importance of the balance between rates of sediment supply to the toe by bank failure and sediment removal by the flow in the channel (Thorne 1982). If the flow is able through time to remove all the debris from bank failures and to continue attacking the intact bank, instability is maintained, failures continue, and the bank retreats rapidly. If the flow is unable through time to remove all the debris then a talus slope develops, buttressing the intact bank and protecting it from further erosion and instability. The bank tends to stabilize and it retreats less rapidly as a result. This sediment balance, described by the "state of basal endpoint control", ultimately determines the retreat rate of an eroding bank, and, in a widening channel, also determines the rate of widening and the eventual width. The concept is equally applicable to a dynamically stable channel which is laterally active. Here the rate of accumulation on the depositional bank and the rate of removal on the eroding bank are matched over time.

It might be possible to incorporate the concept of basal endpoint control into hydraulic models of downstream channel geometry. This would require a sediment budget for the toe area of left and right banks based on the respective rates of sediment supply and removal. The caliber and supply rate of sediment would be estimated from bank stability analyses, while detailed information on the velocity and boundary stress distributions and the local sediment input from upstream would be used to estimate sediment output. If this approach could be used to model channel width control it would represent a significant improvement over empirical techniques.

DATA COLLECTION

The papers presented at this meeting make it quite clear that the data routinely collected for most small catchments in the United States are insufficient for application of quantitative methods of prediction based on geometric or geomorphic analyses, tractive force considerations, or the minimum stream power concept. Neither could mathematical or physical models be calibrated without considerable extra data collection. Despite this insufficiency, the data collected generally by the Soil Conservation Service and the U.S. Geological Survey do allow semiquantitative estimates of the likely direction and approximate magnitude of changes that might be expected. Perhaps the greatest variation to be noted between the results obtained by workers using different semiquantitative approaches is in their predictions of the time scale over which changes will occur and stability be re-established.

Ideally the data collection network should be extended and intensified to include many more catchments and many more variables. In particular, data are usually lacking on sediment transport (especially bed load), and the size, gradation, and degree of armoring of the bed and bank material. The huge cost of collecting such data routinely for many hundreds of catchments means that it is unrealistic to propose that such a program be mounted. A more realistic alternative may be to propose that data collection should continue roughly along present lines but that at an early stage in any proposed water resource scheme a pilot study should be undertaken to identify possible problems. A semiquantitative geomorphic study like any of those reported here of the Elk River would be sufficient to do this. On the basis of the pilot study an intensive data collection program, concentrated on likely problem areas, could be executed in the period between initial planning and the period of major design and decisionmaking some years later. At that stage, state-of-the-art quantitative predictions could be made on the basis of at least some comprehensive data. If the pilot study suggests that serious problems would not arise, as was the case with the Elk River, then the intensive study might not be necessary.

If this strategy were to be adopted it would still be essential that monitoring be continued after construction, though not intensively, so that if unforeseen problems did start to develop they could be detected and dealt with at an early stage. Monitoring of this type should be an integral part of any and every water resource or flood control project, because it enables scientists and engineers to check the accuracy of their predictions and the performance of the project. It could be carried out at a very small cost in the form of perhaps a semiannual official channel inspection, augmented by cooperation and liaison with local land owners, floodplain dwellers, and anglers. Monitoring would then be stepped up immediately if channel instability were detected, to allow time for remedial and preventive measures to take effect before serious destruction of the channel could occur.

There is another and equally important reason to recommend that the pre- and post-construction fluvial and sedimentary regime and the degree of channel instability be studied. This is to develop a wide range of case histories. Case histories from similar projects in other areas were cited by nearly all the participants as an important aid in predicting downstream response. The greater the variety and number of cases documented the better is the chance of finding a good analogy to the case in point, and the more reliable will be the predictions which are made.

CONCLUSIONS

The direction and approximate magnitude of overall downstream channel changes which result from diversion or reservoir construction can be predicted with a good degree of confidence, but there is a need for more research into processes of bank erosion and width adjustment. Detailed local data required for habitat evaluation cannot be obtained from catchment scale models and so a nested approach consisting of hydrologic, sedimentary, and hydraulic models applied at catchment and local scales is needed. If procedures based on natural processes and the natural inclinations of the river are used in developing a river system, many undesirable repercussions from that development can be avoided.

All the data required to apply quantitative prediction techniques to a river system are seldom available. It is unrealistic to hope that this situation will change in the near future. An alternative approach may be to use available data in a semiquantitative geomorphic pilot study at the outset of a project. Areas of potential problems identified in this study could be the focus of a program of more intensive data collection so that at the time of major decisionmaking the different design and management alternatives could be evaluated quantitatively using modern techniques based on at least a few years data.

Data collection at a reduced level and river system monitoring must continue into the post-construction period so that any unforeseen problems can be detected and remedial steps taken before serious damage occurs. Such monitoring is also vital to check the accuracy of the predictions and to build up a library of case histories to aid in future schemes and predictions in analogous situations.

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EVALUATION OF DOWNSTREAM CHANGES FOR THE ELK RIVER

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INTRODUCTION

The study reach of the Elk River is near Longton, Kansas. The question relative to the river is "What will happen to the morphology of the stream channel as a result of the changes in streamflows and sediment discharge caused by the construction of a number of small flood retaining reservoirs upstream of a reach of stream?" Specifically, for both a short and long time after construction of the reservoirs, the meandering pattern, channel configuration, substrate material, and the pool-riffle sequence need to be determined.

At present, materials of the channel consist mostly of fine gravels for the bed and loamy soil for the banks. The banks are lined with trees. The study reach of the river has a channel slope of 0.000462 and a valley slope of 0.00075. The sinuosity of the river, taken as a ratio of these two slopes, is obtained as 1.62. For this flat slope, the channel is not subject to braiding or rapid increase in width.

VARIABLES

The independent variables, dependent variables, and physical constraints for this problem may be summarized below.

a. The independent variables include the water discharge and sediment inflow.

b. The dependent variables include the channel width, depth, slope, sinuosity, sediment size, pool/riffle spacing, etc.

c. The physical constraints include the bank slope, the valley slope, geology of the alluvium, vegetation, etc.

The river banks are lined with dense trees which have played a very important role in developing the present river morphology. Because of the protection and resistance provided by the trees, the river has steep banks and a very small width-depth ratio. These again affect its sinuosity, riffle-pool spacing. Therefore, a change in the tree protection alone will significantly change the river morphology.

The alluvium consists of silt, loam, and gravels. At present, the channel has a gravel bed and silty and loamy banks. Because of the limited thickness of the alluvium, the channel bed has reached the bedrock at several places as evidenced by scattered rock outcrops. For this reasons, physical constraints due to bedrock will limit channelbed degradation to local reaches, if any.

THE APPROACH

In predicting the morphological changes for the river, the regime approach will be employed. There is a never ceasing tendency for a river to attain equilibrium (regime). With the changes of the water discharge and sediment inflow, the river will undergo gradual changes to seek a new regime. These changes are limited by the physical constraints listed above and a new regime may never be attained. However, the regime approach will provide the directions of change for the river.

Regime relations developed previously generally relate the channel width to the bankfull discharge. For example, the Lacey relation for regime canals with a sand bed has the form

$$P = 2.67 \sqrt{Q} \tag{1}$$

The surface width B for gravel streams has the general form

$$B = C\, Q^{0.5} \tag{2}$$

A graphical relation for regime gravel streams developed by Chang (1980) is shown in Figure 1. This relationship was developed based upon a flow resistance formula, a sediment load formula, and the concept of minimum stream power. In this graphical relation, the channel width and depth are shown as functions of the channel slope, the bankfull discharge, and the sediment size. The bank slope plays a significant role in channel morphology. It has been found in the analytical study (Chang 1980) that for a steeper bank slope, the channel has a smaller width and greater depth. On the other hand, a flat bank slope indicates greater width and smaller depth.

The threshold condition in Figure 1 is based upon the threshold condition for gravel movement given by the following equation

$$\frac{\tau}{g(\rho_s-\rho)d} = 0.03 \tag{3}$$

where τ = shear stress; g = gravitational acceleration; ρ_s = density of sediment; ρ = density of water; and d = sediment size.

Above this line, the channel bed is mobile. Below this line, the channel bed is not mobile although "through-put" load may still exist. Based upon Equation 3, the critical channel slope for the threshold condition may be related to the bankfull discharge and sediment size as (Chang 1980)

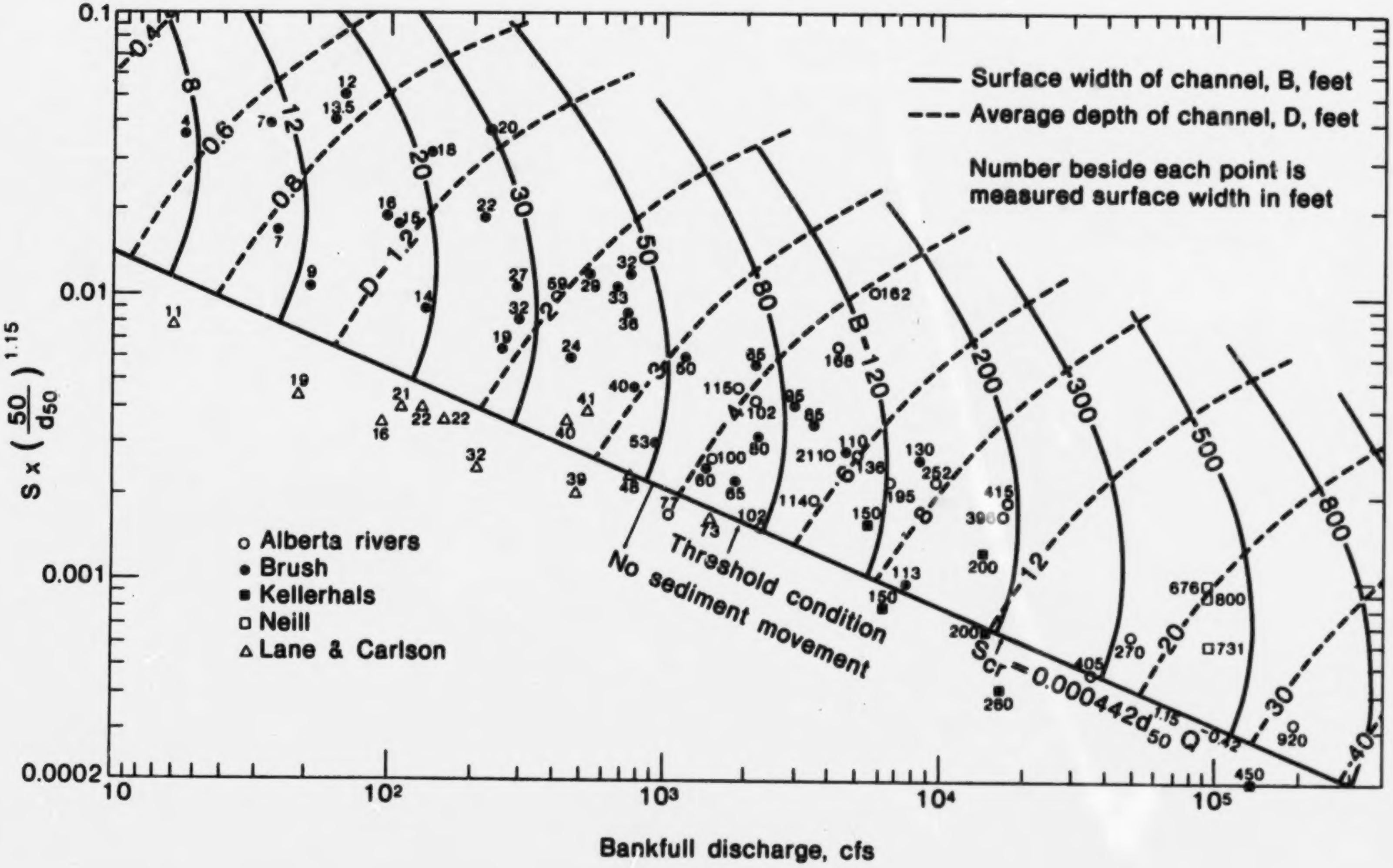

Figure 1. Relation of channel geometry with slope and bankfull discharge for regime gravel streams.

$$S_{cr} = 0.000442\ d^{1.15}\ Q^{-0.42} \tag{4}$$

For the study reach, the bankfull discharge is estimated to be 7000 cfs and the representative sediment size is estimated to be 15 mm. Substituting these values into Equation 4, one has $S_{cr} = 0.000242$. Since this slope for the threshold condition is less than the channel slope, the channel bed is mobile at the bankfull discharge. Using the bankfull discharge of 7000 cfs and $S\ (50/d^{1.15} = 0.00046\ (50/15)^{1.15} = 0.00184$ for the study reach, its position may be placed in Figure 1. The analytical channel width and depth for this point are obtained as 140 feet and 8 feet respectively, considerably less than the actual values. The major reason for the discrepancy is due to the large roughness and the steep bank slope of the study reach in comparison to most other gravel streams.

SHORT TERM CHANGES

Judging from the distances to the reservoirs, short term changes will not affect the study reach. After the completion of reservoir construction, changes will start to occur in channels immediately downstream of the reservoirs and then progress downstream gradually. Being depleted of bedload sediment, the water flowing out of the reservoir will start to scour the channel bed. In the process of scour, finer materials will be removed and transported downstream first. This will result in coarsening of the bed material and formation of an armored layer.

LONG TERM CHANGES

Judging from the distances of the reservoir projects to the study reach, their impacts to the study reach will be of long term nature. The present storage of sediments in the channels leading to the study reach will continue to affect its morphology for quite a long time. With the changes in water discharge and sediment inflow caused by the reservoir projects, the study reach will feel these changes eventually. Basically, it will seek its new regime by adjusting its width, depth, slope, sediment size, and meandering pattern subject to the physical constraints. The adjustments are complicated by the fact that these variables are inter-related. In the following discussion, each individual change will be described first, then, how each change affects other changes will be investigated. The channel will be delicately adjusted in consideration of all the operative variables.

First of all, the regime channel width and depth are functions of the bankfull discharge. With the reduction in discharge, the study reach tends to adjust itself into a smaller channel with a narrower width and shallower depth.

With reductions in both water discharge and sediment inflow, the study reach tends to assume a steeper slope as analyzed below. The reservoir projects will block off 59% of the watershed area resulting in a reduction in bedload inflow to the study reach of roughly 59% in the long run. The reduction in bankfull discharge is more difficult to estimate; it is assumed to be 50% for the convenience of analysis. Now, for regime channels, the sediment discharge is roughly proportional to the square of the bankfull discharge when other variables are constant (Chang 1980), or

$$Q_s \sim Q_w^2 \tag{5}$$

Again, the square power is only a rough estimate. Despite these rough estimates, the trend of variation for the slope can be ascertained. As the bankfull discharge is reduced to half, the sediment carrying capacity is reduced to one fourth. Since the sediment inflow is reduced by roughly 59%, greater transport capacity is required and it will be achieved by a combination of slope increase and other adjustments.

The decrease in channel depth and increase in slope indicate that the study reach is subject to aggradation. The extent of aggradation is not expected to be very significant because of other inter-related adjustments. Specifically, aggradation in the main channel will raise the base levels of the tributaries. The raise will then reduce the amount of sediment inflow from a tributary and the size of the inflow sediment. These again will reduce the aggradation trend of the main channel until a delicate balance is reached.

The increase in channel slope may be achieved by the combination of aggradation and reduction in meandering sinuosity. For this reason, the study reach tends to become less sinuous.

The spacing of the riffle-pool series has been related to the channel width in previous studies (Leopold 1964; Schumm 1977). Since the width tends to decrease, the spacing of the riffle-pool will likely decrease.

One physical constraint that plays a very important role in the present channel morphology is the vegetation on the channel banks. Because of the dense tree protection, the channel banks are steep and the width-depth ratio of the channel is small. A loss of the protection will result in reduction in the channel bank slope, an increase in width and a decrease in depth.

CONCLUSIONS

The following conclusions may be summarized:

1. The present river channel of the Elk River has an unusually large depth and small width-depth ratio. These morphological

features are related to its steep channel banks and high degree of channel roughness. The dense growth of trees has contributed significantly to the steep bank slope and roughness. For this reason, removal of the trees would cause significant changes in river morphology.

2. The study reach of the river near Longton is not subject to any significant short term changes due to the reservoir construction. Short term changes will be limited to river channels in the downstream vicinity of the dams.

3. With the changes in water discharge and sediment inflow, the study reach will experience long term changes. These changes are the result of the delicate adjustments of the channel width, depth, slope, sediment size, meandering pattern, etc. Since these morphological features are inter-related and their adjustments are subject to the given physical constraints, the extent of these adjustments is not expected to be very significant.

4. The study reach is predicted to develop into a smaller channel with narrower width and shallow depth. Its slope tends to become steeper; the channel bed tends to aggrade; its sinuousity tends to decrease; the riffle-pool spacing tends to decrease; the substrate material tends to become less coarse.

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CHANNEL CHANGE WORKSHOP PROBLEM NUMBER 3, ELK RIVER, KANSAS[1]

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ABSTRACT

The Elk River, Kansas is typical of many streams in the Mississippi drainage system. It has a deep channel incised into late to postglacial valley-fill deposits. At present, the distribution and properties of these valleyfill deposits are undefined, and this lack of information prevents reliable prediction of channel adjustments resulting from significant watershed changes. Studies in northern Mississippi and western Iowa, which have established the significance of comparable valley-fill deposits, have been used as case examples. This case history application is based on preliminary results indicating the valley-fill units were deposited in response to paleoclimatic controls. Alternate scenarios are presented based upon vegetative or stratigraphic controls and upon hydraulic-induced or gravity-induced bank failure.

INTRODUCTION

The central theme for this workshop is "What will happen to the morphology of the stream channel as a result of the changes in streamflows and sediment discharge caused by the construction of a number of small flood retarding reservoirs upstream of a reach of stream?" The Elk River, Elk County, Kansas, a gravel-bed river, was assigned and the reach near Longton, Kansas designated for study. The specific questions to be considered are:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material?
4. What will be the pool riffle sequence?

The following data were supplied.

1. General Plan - Elk River Watershed Joint District No. 47,
2. Soil Survey of Chautauqua County, Kansas (Chautauqua County is immediately south of Elk County),

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[1] This manuscript was prepared by a U.S. Government employee as part of his official duties and legally may not be copyrighted under the law of 1 January, 1978.

3. Geology, Mineral Resources, and Ground Water Resources of Elk County, Kansas,

4. Plan, profile and cross section diagrams for the Longton reach of Elk River, Kansas,

5. Monthly streamflow data for Elk River basin, Kansas,

6. Elk River bed material,

7. Elk River sediment yield,

8. Frequency-runoff curve,

9. Unit discharge hydrograph curves and table of values,

10. Evaluation-discharge curves and table of values,

11. Pictures of the Elk River channel at selected locations.

Although this is a sizable data base, additional data and expertise are required to fully answer the four specific questions. Additionally, extended field observations should be made before any attempted evaluation, and such evaluation must consider the Longton reach as an integral component of the entire watershed system. The following discussion is, therefore, presented to stimulate discussion and is not intended to be definitive.

Limitations in the current state of the art for gravel-bed rivers were identified by several of the authors at the recent International Workshop on Engineering Problems in the Management of Gravel-Bed Rivers, Greganog Hall, Powys, Wales. Two major limitations pertinent to definitive solutions of the four questions for Elk River are a) the poor definition of the influence of upland watershed conditions on the movement of coarse sediment to the main channels, and b) the lack of information describing channel bed and bank material stability. Inherently, both the type and magnitude of instability must be considered for this latter influence. Moreover both influences must be evaluated as interactive components of the complete system.

Identification of failure modes and magnitudes for Elk River channel materials is critical for definitive solutions for the four questions, particularly for long term channel reactions. Such data, however, are incomplete. Channel beds are apparently stabilized by hard-rock outcrops, and vertical (thalweg) degradation is improbable. In an attempt to generalize about the significance of bank material stability, I have a) generalized the results of our current channel stability studies in northern Mississippi, b) compared the nature of the northern Mississippi valley-fill stratigraphic units to those of Iowa, and c) noted similarities between Elk River watershed features and comparable features in Iowa and northern Mississippi.

INFLUENCE OF CHANNEL BED AND BANK MATERIAL, NORTHERN MISSISSIPPI

Four valley-fill stratigraphic units influence channel stability and morphology in northern Mississippi. The youngest of these units, termed postsettlement alluvium (PSA), was produced in historic times, resulting largely from cultural activities. It is unweathered and directly overlies either a young paleosol or an old paleosol. The PSA typically varies in thickness from several inches to several feet. The weight of this material functions as a loading term for gravity-induced failure of either of the underlying paleosols.

The next youngest of these units is the young paleosol, a relatively unweathered fluvial deposit. Typically, the basal part of this unit is composed of lateral accretion deposits and the upper part is composed of vertical accretion deposits. The unit was deposited in areas where streamflow had eroded (incised into) an older valley-fill unit. This cut-and-fill was frequently associated with lateral meandering. The young paleosol deposition and weathering commenced about 2,500 ^{14}C years Before Present (B.P.) (Figure 1d). The unit is termed a paleosol because it is buried by modern sediments and because it must be differentiated from present soil classification units. These soil units are materially influenced by the properties of the PSA. Failure of this young paleosol material results primarily from gravity stress, and such failure is accentuated by the development of vertical tension cracks parallel with the bank. This tension crack development is undoubtedly related to the relatively unweathered and, hence, isotropic nature of this silty material.

Old paleosol materials underlie the young paleosol and/or historic sediments. These materials are fluvial deposits and represent a major phase of valley aggradation. They are highly weathered and are typified by a well developed polygonal structure which controls the mode of bank failure. The polygonal seam materials have minimum stability, and block-type failure is induced by gravity stress. In general, the old paleosol materials are more stable than the younger materials. Deposition of this old paleosol material occurred around 10,000 ^{14}C years B.P. (Figure 1d).

Gravel and/or sand deposits underlie the old paleosol. These deposits represent the initial phase of valley aggradation, starting at or immediately prior to 12,000 ^{14}C years B.P., following a major phase of valley degradation. The gravels and/or sands are usually unconsolidated but are occasionally cemented by iron, forming bed-control sills where they crop out in the channels. Exposure of the unconsolidated materials in a bank toe position, resulting from vertical degradation, typically increases rates of failure due to gravity stress.

The present day channel morphology in northern Mississippi has been largely determined by the presence or absence of the iron-cemented bed control sills. Where such sills are absent, such as in Johnson Creek, a tributary of Peters Creek, thalweg lowering has progressed to a sufficient depth to expose the unconsolidated gravels and/or sands in a bank toe position. For this condition, gravity forces become the limiting stress for bank stability. Thalweg degradation started at least 40

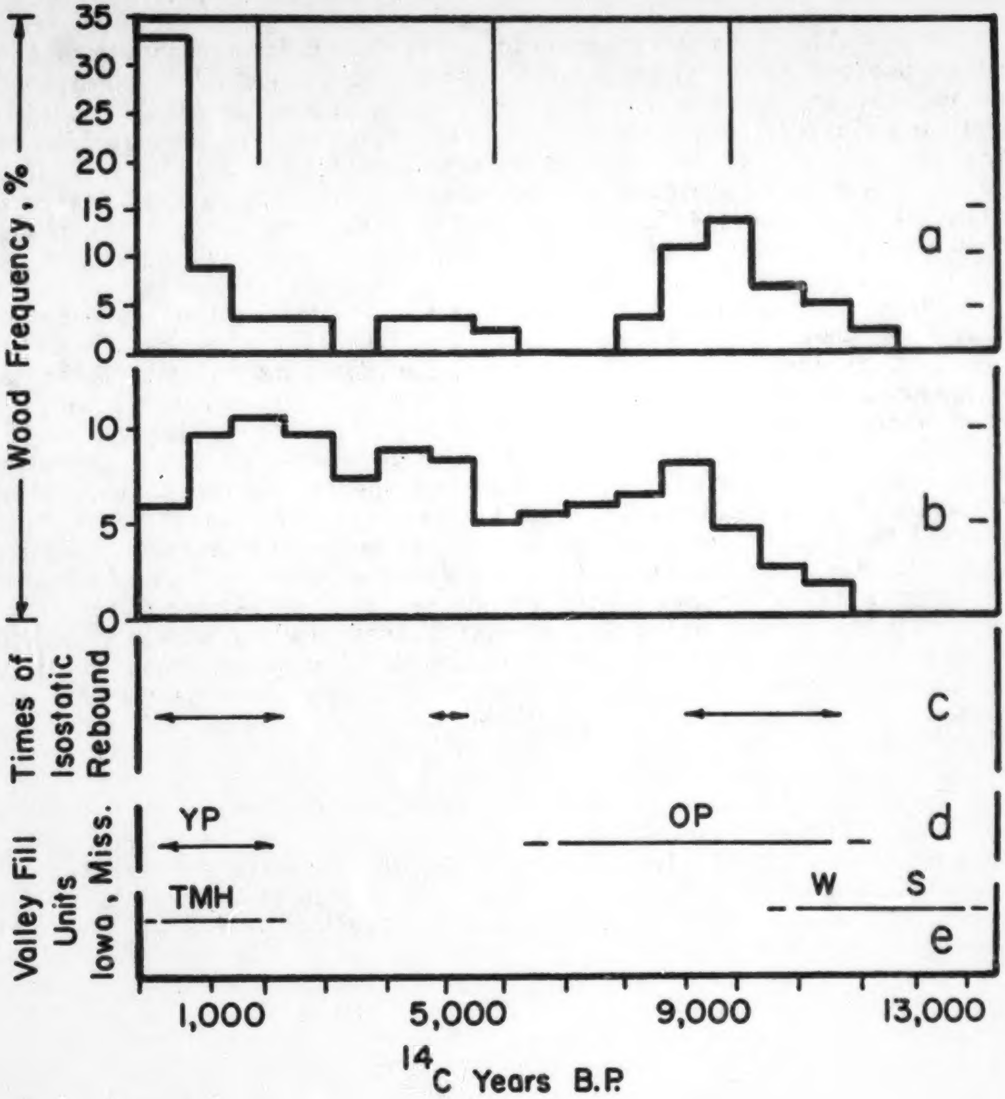

Figure 1. Valley-fill chronology. a) Frequency histogram of ^{14}C dates for 57 samples from northern Mississippi, b) Frequency histogram for 815 ^{14}C dates selected from the journal Radiocarbon (Wendland and Bryson 1974), c) Times of paleclimate-controlled crustal rebound in the Great Lakes area (Flint 1957), d) Paleosol ages in northern Mississippi; YP = young paleosol, OP = old paleosol, e) Valley-fill ages in Iowa; T = Turton, M = Mullenix, H = Hatcher, W = Watkins, S = Soetmelk (Daniels and Jordon 1966).

years ago and has progressed upstream in the form of either a diffuse or discrete headcut. Channel width-to-depth ratios are coherent upstream of the headcut where the channel bed material is either of the cohesive paleosols (Figure 2). Downstream of the headcut, however, Johnson Creek is a sand-bed channel and measured channel depths are less than scour depths. Width-to-depth ratios are random downstream of the headcut, resulting from excessive channel widening particularly in young paleosol materials (Figure 2). Where iron-cemented sills have prevented vertical degradation, such as in Goodwin Creek, a gravel-bed tributary of Peters Creek, excessive lateral channel erosion has occurred (Figure 3). This widening is not constant throughout the system but appears to be associated with local stratigraphic and/or channel morphometric conditions. Channel width-to-depth ratios are random and channel sinuosity is highly variable. Selected reaches in both channels have widened excessively since 1968.

In summary, the energy expenditure within the channels has not been uniform over relatively long channel lengths but has been concentrated in relatively short reaches of the channel. As illustrated for Johnson and Goodwin Creeks, this form of degradation is intimately associated with the nature and distribution of the valley-fill stratigraphic units of Holocene age. Channel morphology and channel morphometric changes are similarly intimately associated with these Holocene units. Gravity stresses limit bank stability for channels which are presently incised and the magnitude or rate of failure ultimately depends upon the ability of the flow to remove the slough from the bank toe position. Slough from either of the paleosols and from the postsettlement alluvium is easily removed, generating the failure process.

COMPARABILITY OF VALLEY-FILL SEQUENCES IN IOWA AND NORTHERN MISSISSIPPI

Daniels and Jordan (1966) have identified six valley-fill units in Harrison County, Iowa. These six are recent (postsettlement) deposits and five members of the DeForest formation including the Turton, Mullenix, Hatcher, Watkins, and Soetmelk members. Wood contained within the latter two members has been dated at 11,120 to 14,300 ^{14}C years B.P. The comparable ages of the Turton, Mullenix, and Hatcher members ranged from less than 250 to 2020 ^{14}C years B.P. These ages (Figure 1e) are in general agreement with ages of the two paleosols identified in northern Mississippi (Figure 1d), and are in general agreement with the periods of muck accumulation in closed systems in Iowa (Ruhe 1969).

In addition Ruhe (1969) has identified two erosion surfaces in Iowa which, preliminary data show, may have counterparts in northern Mississippi. The Wisconsin erosion surface which developed in Iowa prior to 14,000 ^{14}C years B.P. is comparable to the observed erosion cycle in northern Mississippi which developed prior to 12,000 ^{14}C years B.P. In addition, the Iowan erosion surface formed prior to 18,300 ^{14}C years B.P. (Ruhe 1969) and may be comparable to an erosional surface in northern Mississippi overlain by organics which are 17,000 ^{14}C years old. This surface in northern Mississippi has not been fully defined. Other similarities include the ages of older alluvial fills ranging from 37,600 ^{14}C years B.P. in Iowa (Ruhe 1969) to 34,900 ^{14}C years B.P. in

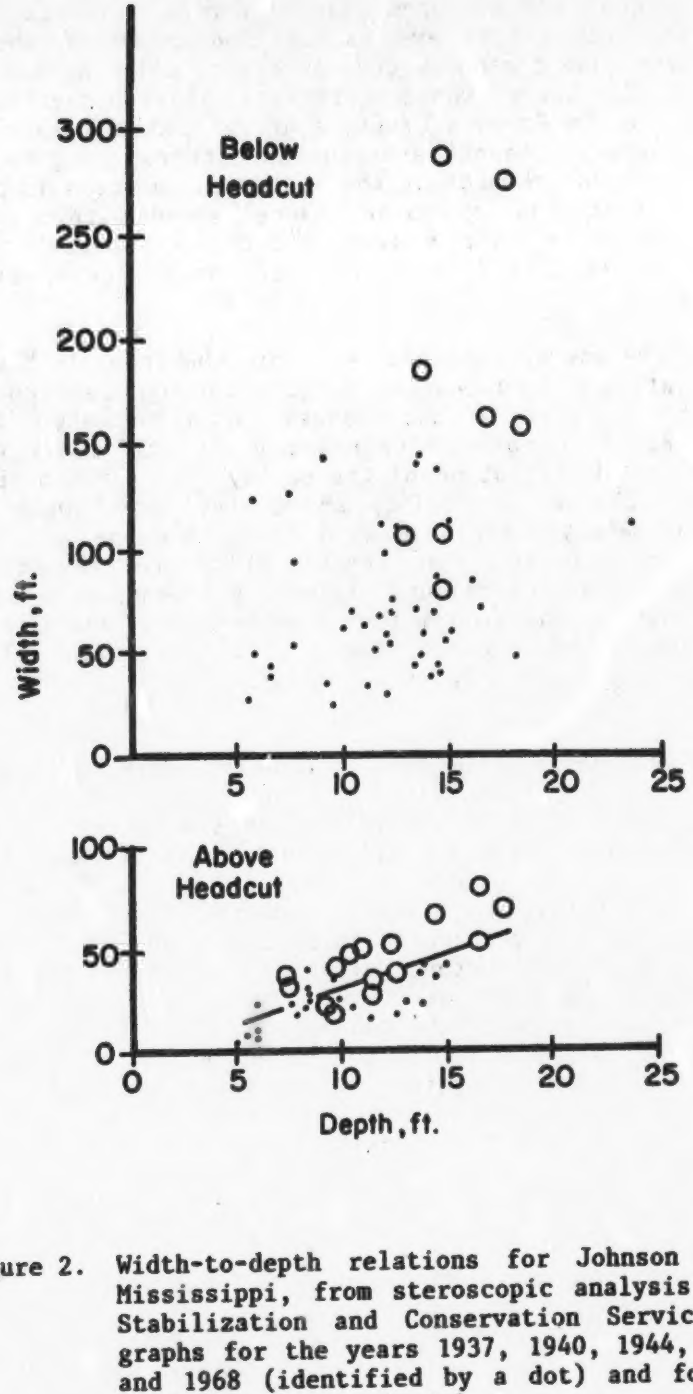

Figure 2. Width-to-depth relations for Johnson Creek, northern Mississippi, from steroscopic analysis of Agricultural Stabilization and Conservation Service aerial photographs for the years 1937, 1940, 1944, 1953, 1957, 1963 and 1968 (identified by a dot) and for the year 1975 (identified by a circle).

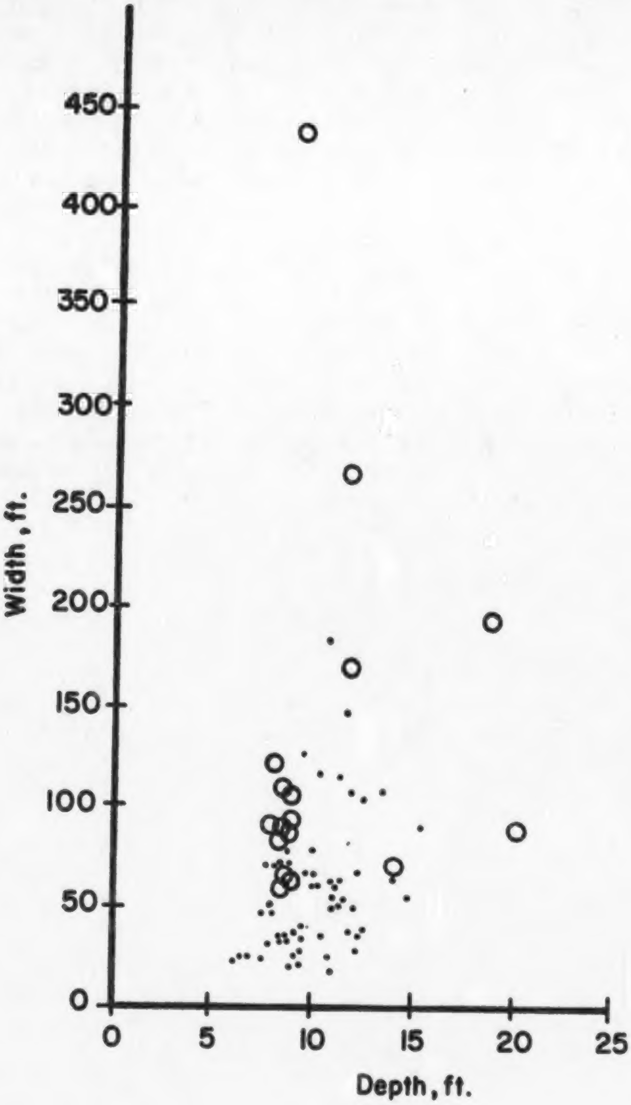

Figure 3. Width-to-depth relations for Goodwin Creek, northern Mississippi, from stereoscopic analysis of Agricultural Stabilization and Conservation Service aerial photographs for the years 1937, 1940, 1944, 1053, 1957, 1963 and 1968 (identified by a dot) and for the year 1975 (identified by a circle).

northern Mississippi (Grissinger). These dates illustrate the dynamic nature of the fluvial systems during the late Quaternary, especially during the Holocene.

More significantly, the hypothesis that paleoclimate has been the dominant control of the late Quaternary fluvial systems and valley-fill processes is supported by the agreement among a) the (wood) age frequency for northern Mississippi (Figure 1a), b) the age frequency compiled by Wendland and Bryson (1974) for paleoclimatic interpretation (Figure 1b), c) the times of paleoclimate-controlled crustal rebound in the Great Lakes area (Figure 1c) identified by Flint (1957), and d) the ages of the valley-fill sequences (Figure 1d and 1e). Various other comparisons supporting climatic control of fluvial episodes are discussed by Knox (1975). Verification of this hypothesis together with channel bank stability studies complementary with those in northern Mississippi (but in other areas) will facilitate prediction of channel response for "uncalibrated" areas. Such stability studies have been reported by Bradford and Piest (1980) for western Iowa. Their results are comparable with ours for northern Mississippi in that a) gravity forces are the limiting stress for bank stability and b) stability is controlled by the distribution and properties of the valley-fill stratigraphic units. For the subsequent discussion, I have assumed that a) the hypothesis of paleoclimatic control is valid and b) the western Iowa and northern Mississippi studies are viable "calibrations" for Elk River channel stability problems.

ELK RIVER WATERSHED

Not enough data are available to positively relate the valley-fill sequence in Elk River watershed with the sequences in either western Iowa or northern Mississippi. Bayne (1958) reported that some of the valley-fill units may be Kansan or older whereas the valley units in surrounding counties are reported to be mixed Illinoian-Wisconsin (Jamkhindikar 1967) or mixed Wisconsin-Recent (Holocene) (Frye and Leonard 1952) deposits. The parent material for the Verdigris, Brewer, and Osage soils is identified as alluvium in Butler County (Soil Conservation Service 1975) and is assumed to be Holocene in age.

The Verdigirs (Cumulic Hapludolls) soil occurs adjacent to present stream channels, is well drained, has no B horizon development and is friable (when moist) to a depth of 57 inches. These properties are comparable with those of soils formed in PSA or young paleosol materials in northern Mississippi. The Brewer (Pachic Argiustolls) and Osage (vertic Haplaquolls) soils are moderately well to poorly drained, frequently have B horizon development and are firm to very firm (when moist) at a depth of 30 to 66 inches. Similarly, these properties are comparable with those of soils developed over old paleosol materials in northern Mississippi. The sandy to gravelly basal phase of this valley fill, the presence of older fill materials preserved as terraces, together with the preceding soil similarities, all suggest that the Holocene valley-fill sequence in southeastern Kansas is similar to that in western Iowa and northern Mississippi.

Two features of the present day Elk River channel are pertinent to the (assumed) role of the valley-fill stratigraphic materials. These features are the apparent incision of the channel to a depth of bedrock control, about 30 feet, and the apparent bank stability. Most cross sections show two bank slopes, an upper, relatively low angle bank slope and a lower, steeper bank slope. I interpret this change in bank slope to indicate past thalweg lowering with a change in channel cross section shape from a relatively broad channel to a deep channel. This past thalweg lowering may have resulted from historic land use changes which either directly increased the rate and magnitude of runoff or indirectly resulted in the same change in runoff due to excessive denudation (erosion) of the upland areas. One critical area of uncertainty is the nature of the lower bank material. The present lower banks may be composed of _in situ_ materials of sufficient cohesion to resist detachment or they may be composed of slough materials from overlying deposits which have been stabilized by vegetation. Several of the pictures show trees, estimated to be 25-35 years old, close to the channel and the aerial photograph shows more or less continuous timber bordering the channel. This vegetative cover and the lack of meander (lateral) migration indicates stability of the channel system for an extended period of time. Exceptions to this vegetative cover are between ranges 11-1 and 3-5 where the channel apparently has been relocated, and on the outside bank of the meander at range 13-2. Regardless of whether the channel system has incised into relatively resistant material, possibly similar to the northern Mississippi old paleosol materials, or has been stabilized by vegetation, it is evidently stable at present.

An additional area of uncertainty concerning the proposed watershed project is the influence of the structures upon the sediment load. The proposed structures will retard the runoff from 58.6% of the watershed area. According to the sediment yield curves, these structures will reduce the bedload yield by 60% and the suspended sediment yield by 57% for this 409 square mile watershed. I am somewhat surprised that these estimated reductions are so similar. Nevertheless, these structures will influence peak discharge and will produce sediment starvation which, individually or combined, may influence channel behavior in two ways. Channels immediately downstream of the structures may degrade depending upon the relative degree of sediment and flow reduction and upon reach conditions such as bed slope, channel roughness, bed and bank material size, etc. Such degradation, if it does occur, will probably be relatively insignificant in influencing Elk River channel behavior. An additional and more significant influence may be the accelerated degradation of, and sediment production by, tributaries not directly controlled by structures. This type of accelerated degradation of uncontrolled tributaries frequently results from mainstem stage reduction. Depending upon reach conditions, the relatively coarse sediments, produced by such a mismatch of peak stage, may be transported to the Elk River channel and deposited as tributary fans. These fans would influence secondary flow, possibly producing local problems of bank instability. Quantification of these features would require sediment routing data based on individual sediment sizes and a thorough knowledge of the distribution of these sediment sizes in the watershed.

The second set of possible influences of the proposed structures involves the direct effects on Elk River channel of peak-discharge reduction and sediment reduction. Prolonged, within-channel flows may impair the vegetative cover on the banks due to prolonged submergence. If the present bank stability is a result of vegetative cover, then any vegetative reduction could impair bank stability. Additionally, prolonged, within-channel low flows may excessively erode the bank toe materials. The Elk River channel thalweg is nonuniform and, for such channels, low-flow point shear velocities may be an order of magnitude or more greater than average shear velocities. Toe instability could result if such point velocities were imposed on bank-toe materials for long periods of time. Such toe instability would critically affect overall bank stability.

If toe instability occurs, gravity-induced failure will become the dominant mode of channel instability. This is really a two-stage process involving a) failure due to gravity stress, and b) removal of the slough material by stream flow. Prolonged within-channel sediment-deficient high flows, sufficient to remove slough material, would regenerate the failure process, leading to acute bank instability problems. The probability of this type of acute instability developing in Elk River is unknown. I assume it is possible due to a) the longer duration of within-channel flows resulting from structural control of the present flooding and b) the prevalence of gravity-induced bank failure in both western Iowa and northern Mississippi. Detailed analysis of the valley-fill stratigraphic materials must be made to better define the possibility or probability of this scenario.

WORKSHOP QUESTIONS

As stated previously, the preceding discussion is not intended to be definitive but is presented to stimulate discussion. It is tempting to directly apply regime-type equations to the subject questions, but I have refrained from this exercise since I do not consider Elk River to be a true alluvial channel. The bed is controlled and the banks are not "free to adjust" as evidenced by their past and present stability.

Based on the apparent stability of the present Elk River channel, I believe that the proposed structural control of flooding will not result in any significant channel adjustment in the short term. Minor aggradation may occur at tributary confluences but this feature should be relatively inconsequential. In the long term, bank stability will determine the changes in channel morphometric characteristics; such changes will be site specific and inherently nonregime in nature. Quantification of the possible changes will depend upon the relative significance at each site of vegetative and stratigraphic controls relative to hydrologic erosive conditions, particularly those associated with nonuniform flow. These conditions and controls are not presently defined.

Three possible scenarios are outlined for long term stability and, for each of these cases, bank toe stability is the critical condition. These scenarios are: a) the bank toe will be stable, b) the bank toe will be unstable and bank failure will depend primarily upon hydraulic stress and c) the bank toe will be unstable and bank failure will be progressive, resulting from gravity induced failure with subsequent removal of the slough material by high-stage flow. None of these possible scenarios would result in regime-type channel changes.

Insignificant channel adjustment will result from implementation of the flood control program if the bank toe materials remain stable. More pronounced instability will occur if the toe material is unstable; the degree of bank instability will depend upon stratigraphic controls.

If the present banks are composed of _in situ_ cohesive valley-fill materials, the channels will widen but at a relatively slow rate. Timber falls into the channel could create excessive secondary flow conditions leading to accelerated bank instability. As channel widening progressed, pool depths could become somewhat shallower and the substrate materials would become somewhat finer. The riffle-pool sequence and the sinuosity pattern would be unaffected, however. In this scenario, the vegetative cover is a result of the relatively large resistance to erosion of the stratigraphic materials. Channel width would approach a "limiting width", beyond which the imposed flow would be unable to cause detachment.

The worst possible scenario is that the present banks are composed of slough materials, residual from some past cycle of bank instability. In this scenario, the present stability is not related to stratigraphic controls, but to vegetative controls introduced in the past during a short period of atypical climatic conditions. Removal of the present toe materials resulting in exposure of weak stratigraphic materials would exceed the stability threshold and Elk River channel could "fall apart." Due to the present excessive depth of this channel, gravity forces would become the dominant bank stress, and the "limiting width" would depend upon the capacity of the flows to remove the slough materials. In this scenario, the influence of large storms would become progressively more significant. The channel would widen, possibly to the extent of forming a new floodplain at a lower elevation than the present one. If such a "rejuvenation" was allowed to reach completion, a new regime would be established. For this scenario, quantitative prediction of the channel characteristic changes is not possible since the distribution and properties of the valley-fill units are unknown. Certainly the channel width-to-depth ratio would increase (especially since the channel depth would be reduced). Meander size would probably decrease due to the reduction in within-channel peak flow and due to physical constraints by the new terraces (the present floodplain) but the change in sinuosity is unknown. Substrate material would become finer due to reduced flow velocities and the pools would fill and become shallower. The ultimate pool-riffle sequence is unknown. I must reiterate that the preceding scenario is the worst possible case. Such a scenario would be disastrous, potentially too disastrous to ignore. Channel reconnaissance should be performed periodically (every few years) to verify bank toe integrity and completeness of vegetative cover. Possibly this reconnaissance should be performed as an integral part of a case history documentation for Elk River. Such documentation would be invaluable for future planning in this or adjacent areas.

The preceding discussion of this workshop problem illustrates the significance of valley-fill stratigraphy. For predicting channel changes or general landscape reaction, we must understand the system in the field. Our detail of knowledge of these systems must be commensurate with the problems with which we are concerned. For predicting channel behavior, we must have a detailed knowledge of the distribution and properties of the valley-fill units. For this purpose, the process (formative) controls of the system must be established. Our preliminary studies have indicated that paleoclimate is the process control for the Holocene valley-fill sequence in northern Mississippi and possibly a large part of the Mississippi River drainage area. If this result can be verified, it would greatly simplify solutions to problems such as those posed to this workshop.

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PRELIMINARY ASSESSMENT OF CHANNEL CHANGE, ELK RIVER, KANSAS

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INTRODUCTION

This paper considers the geomorphic response of the Longton reach of the Elk River to anticipated modifications in stream flow and sediment load as a result of construction of a number of small flood retention reservoirs in the upstream watershed. Within the constraints of data and time for problem analysis, only a preliminary qualitative assessment of response is attempted, but a methodology is outlined that will provide both quantitative results and a reasonable prediction of anticipated response. The approach has been applied to river systems as large as the Upper and Lower Mississippi River and as small as ephemeral arroyos in New Mexico, with axcellent results. The methodology is currently being applied to an analysis of the response of the Cochiti to Isleta reach of the Rio Grande in New Mexico to the construction of the mainstem dam at Cochiti in 1973. As results of this latter study are pertinent to the Workshop theme of downstream river channel changes from diversions and reservoir construction, they will be available during the general discussion session of the Workshop to illustrate the methodology outlined in this paper.

METHODOLOGY

The approach recommended for analysis of Elk River response has been described in detail and applied to the Upper Mississippi River in a reference document, "The River Environment," prepared for the Fish and Wildlife Service, Twin Cities, Minnesota by Simons, Lagasse, Chen, and Schumm in 1975. It was refined further relative to an "Assessment of Geomorphic Response of River Systems to Hydraulic Structures" in a paper I prepared for an International Symposium on "Environmental Effects of Hydraulic Engineering Works," held at Knoxville, Tennessee, 12-14 September 1978. The brief outline of the methodology which follows is extracted from these references.

Depending on the data and resources available for analysis the problem of response of the Elk River to the construction of a number of small hydraulic structures should be approached in three phases.

1. A qualitative analysis based on general geomorphic parameters.
2. A quantitative analysis based on specific geomorphic and hydraulic data.

3. A mathematical model of watershed and channel processes in the reach or system of concern.

As listed, each phase requires an increasing commitment of resources, but individully each phase yields meaningful results. These range from a purely qualitative assessment of trends to the numerical results and predictive capability of physical process computer modeling. When applied sequentially this multiphase approach constitutes a powerful methodology for the evaluation of short and long range response of river systems to development.

To the extent that data permits it is desirable to establish, first, the morphologic and hydraulic conditions of the river or reach under consideration before man's intervention. Conditions on the "natural" river form a baseline against which the impacts of man's activity can be assessed. Unfortunately, the historic record and data are usually not sufficient to establish a complete picture of the natural river, except on major systems such as the Mississippi or Rio Grande. As a minimum, it is usually possible to reconstruct the history of engineering activity on a river or reach of concern. Although data on the Elk River provided for the Workshop do not provide a complete picture of engineering activity in the watershed, indications are that such information could be developed with a minimal investment of research effort.

A qualitative analysis of geomorphic response should include an examination of the river in planform, longitudinal profile, and cross section. Where data are available for different time periods (for example, before and after construction of a dam), this analysis produces a time-sequenced picture of morphologic change in three dimensions which can be correlated with the history of engineering activity in the study reach. This correlation provides a qualitative assessment, in terms of trends, of the impacts of man's activity in the reach. In systems that have experienced multiple development techniques (dredging, dikes, navigation dams, levees) an attempt can then be made to isolate the system response to a particular activity of concern, or to predict response to hydraulic structures and development measures being considered.

Township plats normally provide the earliest accurate planform data on a river system. Comparison of these with later topographic surveys, aerial photographs, or the current USGS quadrangle sheets establishes the degree of bankline stability in a system. Even within relatively stable banklines most alluvial rivers exhibit changes in bankline and the number, location and configuration of bars and islands. As these features influence resistance to flow and act as controls in a river reach, an understanding of their evolution is imperative.

Time-sequenced comparison of selected reaches and measuring and comparison of river widths, island area, and river bed area, all provide useful data for establishing and evaluating morphologic characteristics of an alluvial river. Graphical, tabular, and plan view representation of this data provide insight into the evolution of alluvial processes of a particular region and their relation to man's development in the region. The impact of such development techniques as bankline stabilization, contraction dikes, and jetty fields is usually quite apparent in the planform comparisons.

While longitudinal profiles are normally not directly available, they can be constructed from cross section data derived from hydrographic surveys. Comparisons of profiles can be made using either the average depth of the cross section or the thalweg depth, that is, the deepest point in the cross section. Evidence from analysis of the Upper Mississippi indicates that thalweg bed elevations, in general, vary in the same manner as average river bed elevations, and so provide a good, readily obtainable indicator of trends in bed elevation. Both tabular compilations and time-sequenced longitudinal profiles are useful in establishing trends in aggradation or degradation.

The tabulation and plotting of cross section data reveal changes in such morphologic parameters as surface width, bed elevation, average depth, and thalweg position. Cross sections can be established at equal intervals throughout a reach or can be concentrated in those portions of the reach that planform analysis has shown to be morphologically active. Comparison of cross sections adds a third dimension to the qualitative analysis and normally provides the clearest indicator of the impact of such stabilization measures as revetment, contraction dikes, and jetty fields.

In correlating the planform, longitudinal, and cross-sectional data accumulated by these and other techniques, it is not uncommon to encounter apparently conflicting indicators. Here, an understanding of the natural morphologic conditions of the system provides an essential baseline for interpreting apparently anomolous behavior. Efforts directed at synthesizing the geologic history of the region and identifying existing geologic controls, as well as the historical research required to provide clues as to the natural conditions of the river system, often pay unexpected dividends in this regard.

While a qualitative analysis, alone, will yield meaningful results, refinement of conclusions resulting from such an analysis and a more precise assessment of trends as well as a predictive capability can be derived from both quantitative analysis and mathematical modeling. It should be noted, that calibration of a mathematical model involves evaluation and modification of supplementary relations to the basic process equations using field data or theory so that the model will reproduce the historical response of the modeled river system. Establishing the trends in geomorphic components which constitute this historical response by a qualitative analysis, then, provides an essential base for model calibration.

The conventional or traditional quantitative techniques of river engineering are well known and do not require discussion here, however, a few general comments on these techniques in relation to geomorphic analysis and mathematical modeling may be appropriate. Quantitative techniques include using unit hydrographs for water routing and yield from watersheds, the Universal Soil Loss Equation for estimating erosion from watersheds, time-lag methods for flood routing in the channel, sediment rating curve techniques for estimating deposition in reservoirs, and developing relationships for hydraulic geometry of the reach under study. These techniques can be applied to only a relatively small number of conditions and alternatives because of cost limitations for most studies. It is often difficult to predict the response of a system to development alternatives using these methods as they are based on the assumption of homogeneity in time and space. While this approach may be feasible for selected reaches, application on a system-wide basis quickly leads to unmanageable computational requirements. Such dynamic features as degradation and aggradation are difficult to account for, and integrating the subsets of a traditional quantitative analysis into a basin-wide system analysis presents complex problems. The traditional quantitative techniques are, however, extremely valuable in either refining or substantiating the conclusions of a qualitative analysis, or in providing a basis for mathematical model calibration.

To conduct an analysis with a physical process computer model, several major tasks must be accomplished. The first is to develop a mathematical model for routing water and sediment from the watershed and through the existing and modified channel and reservoir systems. This model could be used to determine flow lines and identify areas where excessive sediment aggradation and degradation may occur, as well as to indicate major sources of sediments that flow from tributary systems to the main channel. With such a comprehensive model developed and verified, the second task is to evaluate various operational alternatives or development scenarios. The final task involves selecting an optimum plan for the development or operation of the river basin considering flood control, sediment control, minimizing environmental impact, maximizing water salvage, and other factors of concern for a particular region.

The detailed development of such a mathematical model involves the following steps: data assembly and inventory; data evaluation; development of a data storage and retrieval system; collection of required data that cannot be synthesized; identification of data gaps and synthesis of additional data required for analysis; overall system design including spatial and temporal design, subsystem model development such as the main-stem model, tributary models and watershed models; validation and linking of the subsystem models; development of applicaton data files; model calibration and validation; application of the models to evaluate system response for different design alternatives; and finally, conducting a detailed analysis of selected alternatives.

ANALYSIS

If the analysis of the Elk River were to concentrate on only the Longton reach, that is the 8 river miles between Sections 11-15 and 3-3, it is doubtful that much more than a qualitative geomorphic assessment of response to the proposed flood retention reservoirs could be justified. Selected aspects of the trends revealed by this geomorphic analysis could be verified by quantitative calculations. However, if a complete analysis of the response of the Elk River system to the construction of the 48 floodwater retarding structures is desired, the application of physical process computer modeling would be required.

With the data and time presently available, and considering just the eight miles of the Longton reach, only a preliminary qualitative assessment of geomorphic response will be attempted here. This should provide a basis for more detailed analysis and discussion during the Workshop.

Control of floodwater run off from 59% of the Elk River drainage area and retention of the expected 100 year accumulation of sediment from this area will induce geomorphic change along the Elk River, including the Longton reach. On each of the many tributaries with retention dams one would expect some tendency toward the classic response of a river to dam construction. As a result of clear water release local effects below each dam could include local scour at the dam, channel degradation below the dam, and possible bank instability if significant degradation takes place. The downstream effects could include degradation and bank instabilities along the entire reach between the retention structure and the junction of the tributary with the Elk River. Over the short term this could produce an increase in sediment supply from these tributaries to the mainstem, even though the dam might trap virtually all the sediment produced from the upstream watershed. The magnitude of any short term increase in sediment load depends on resistance of each tributary reach to degradation. This resistance would come from bed and banks of cohesive material, bed rock or geologic control, or armoring of alluvial reaches of the tributary.

As each tributary adjusts to regulated flows of water and sediment over the long term, and cohesive materials, or bed rock, or a well developed armor layer become sufficient to resist scour by regulated flows, a net reduction in sediment supply to the Elk River should be anticipated. (Note pre- and post-project estimates of Elk River sediment yield in the data package.) The magnitude and duration of any short term increase in sediment supply from the tributaries following construction of a retention dam and the development of a stable channel in the reach below the structure will depend on individual channel and watershed characteristics. Based on the characteristics of bed, bed material, and armor layer of the Elk River cross sections described in detail in the data package, and information in the soil survey of Chautauqua County, any initial increase in sediment load from the tributaries should be of short duration, and degradation in tributary reaches below retention structures should be limited.

For the Elk River mainstem, the cumulative effect of this sequence of geomorphic change on multiple tributaries could be significant. There is potential for both short and long term change in meander pattern and planform configuration, composition of the bed, and the riffle/pool sequence along the Elk River. With specific reference to the Longton reach the magnitude of this change will depend, to some extent, on the ability of the channel to absorb and redistribute any short term increase in tributary sediment load, but primarily on the resistance of the reach to a long term tendency toward degradation because of reduced sediment inflow from the tributaries.

The characteristics of the bed, bed load, armor layer and banks described for Sections 11-15 to 3-3 of the Longton reach indicate that this reach should be quite resistant to both degradation and planform change through bank erosion. Of the sections described in detail most show a geologic control (limestone, rock, shale) either exposed in the section or a few feet below an armored alluvial bed. For sections such as 3-1, 3-3, and 11-9, where the bed material is apparently alluvium, the resistance to degradation will depend on development and stability of an armor layer. With indications of an armor layer at 7 of the 13 sections described in the data package, there is sufficient evidence to assume that the channel bed contains the necessary gradation and quantity of coarse material to produce armoring as degradation progresses. However, the response of the Elk River to changes in discharge of sediment and water on multiple tributaries will be so complex that a quantitative determination of the actual amount of degradation to be anticipated in alluvial reaches would require physical process computer modeling of the watershed, tributary, mainstem system. A quantitative approach, short of modeling, would probably not produce an estimate any more reliable than that derived from a purely qualitative assessment. From the morphologic characteristics of the Longton reach one would expect no more than 2 or 3 feet of degradation before either bed rock control or armoring produces stability against the reduced flood peaks of controlled flows.

There are several techniques in the literature for computation of river bed degradation as a result of altered sediment regime, but the assumptions required in their application to a field situation are quite limiting. For example, Komura and Simons developed a technique for calculating "River-Bed Degradation Below Dams" in 1967 but the assumptions required in the numerical example include (with an indication of applicability to the Elk River problem):

1. Sediment transport is completely arrested by the dam (OK),

2. River banks are not erodible (No),

3. Seasonal variations in discharge and temperature of water do not occur (No),

4. Sediment injections by tributaries do not occur (No), and

5. Meandering and growth of vegetation do not occur (No).

Similarly, the USBR "Design of Small Dams" contains an approach for estimating degradation and armoring which places heavy reliance on "engineering judgment" and limiting assumptions.

In the case of the Longton reach of the Elk River the key factor in the analysis must be the influence on the mainstem of altered tributary flow conditions. As with the Komura and Simons approach, most quantitative techniques for estimating degradation and stability through armoring cannot handle this complexity. Important tributaries to the Longton reach include: Wildcat Creek - 2 miles above Section 11-15, Clear Creek - .5 miles above Section 11-15, Hitchen Creek at Section 11-1, Painterhood Creek below Section 3-1; and several smaller unnamed tributaries in the vicinity of Sections 11-11 and 11-15. Under "natural" flow conditions (prior to the construction of flood detention dams) one would expect that slope and sediment inflow of these tributaries would be adjusted to the existing base level of the mainstem. While delta deposits from flood flows on the tributaries might temporarily divert or control the base level of the mainstem, these deposits would normally be redistributed during flood flows on the Elk River proper.

Under post-project conditions on the Elk River (as illustrated by the unit discharge hydrograph in the data package) the tributaries could assume a dominant role in controlling base level. This would be particularly true during any initial period of degradation below structures on the tributaries, when regulated flows on the mainstem may be incapable of moving either the size or volume of material deposited in tributary deltas.

As a case in point, on-going analysis of the response of the Rio Grande to the construction of the mainstem dam at Cochiti reveals that degradational processes have been far more complex than would be predicted by available quantitative techniques. The "classic" degradational wedge, deepest at the dam and tailing out at some downstream geologic control, has not developed. Instead, the initial 8 miles below the dam have shown remarkable stability, apparently because of the inability of regulated flows to move the size or volume of material in numerous arroyo deltas in the reach. The availability of significant quantities of gravel in these deltas has resulted in development of a stable armor layer in the reach. Downstream, beyond the influence of these arroyos, 6 to 8 feet of degradation has occured.

Over the short term, then, planform change could be expected as a result of diversion or blocking of the Elk River at tributary junctions. With time, redistribution of this material could alter the existing riffle and pool sequence as "slugs" or waves of deltaic sedimentary material are moved through the system. High flows will be comparatively rare, and extended periods of low flow will scour the crossings and fill the pools of the existing meander sequence. As tributaries adjust over the long term to altered flow conditions a period of degradation can be expected in reaches such as the Longton reach. A fairly recent (natural?) cut off of a meander bend is evident below section 11-11. Degradation could induce bank instability with a potential for additional meander loop cutoffs at several locations in the reach. This would produce a radical change in slope, velocity, and transport capacity and, again, alter the riffle/pool sequence. However, degradation should be limited in most sections of the Longton reach to no more than 2 to 3 feet by either geologic controls such as the limestone ledge at Section 13-2 (photo 25-2) or development of an armor layer. Once developed, an armor layer should be sufficient to provide stability under conditions of reduced main stem flows. There are several locations in the Longton reach (Sections 11-15, 2180, 11-1, 3-5, 3-4) where degradation could expose rock, shale, or limestone, thus altering the existing substrate material as well as modifying the riffle and pool sequence.

SUMMARY

In summary, there is potential for geomorphic change in the Longton reach of the Elk River as a result of construction of flood retention structures on tributaries. While a qualitative assessment can indicate possible trends (aggradation/degradation, stability/instability) with reasonable assurance, a complete quantitative assessment of response would require application of the multiphase approach outlined at the outset of this paper. Response to altered conditions of water and sediment flow on 48 tributary reaches and 59% of the drainage area will be complex, and will require computer modeling to refine initial conclusions derived from qualitative analysis.

Although the data package provided for this analysis was reasonably complete, additional information concerning bank material and bankline stability would have been useful. With the Elk River system "fixed" in the vertical dimension by numerous geologic controls, bankline stability becomes the crucial indicator of geomorphic stability in terms of the meander pattern and the riffle and pool sequence. Along these lines, a site visit must be considered an absolutely essential element of analysis for any river related problem. Results of any analysis, even at the qualitative level, must be considered tentative until the analysis is supported by at least one day in the field on-site. For example, the preceding qualitative assessment assumed that bed rock control was dominant in the Longton reach, that the tributaries were similar to the mainstem Elk River in terms of geologic controls, alluvium, and bankline vegetation, and that active measures to insure the integrity of bankline vegetation were in effect or planned. If a site visit demonstrated that any of these assumptions was in error, the conclusions of this preliminary qualitative assessment would require revision.

An additional technique that I consider essential for analysis of response in any river system is a literature search for well documented case studies of river response to similar engineering activity in related physiographic settings. This is particularly important where conclusions or projections of response are required using only a limited data base, as was the case with the Elk River. For example, Haung (1977) analyzed the response to large impoundment structures of seven major streams in Kansas. Qualitative methods of fluvial geomorphology as outlined in this paper and hydraulic engineering techniques (quantitative methods) were applied to the problem of river response, to include analysis of the Fall River just to the north of the Elk River watershed. While response to a single large impoundment structure on each river sysem was analyzed, several conclusions derived from Haung's study are of interest in anticipating response of the Elk River system to multiple small impoundment structures. Degradation below the impoundment structure was experienced on all streams considered by Huang, and in all cases the stream below the structure tended to form a relatively narrower and deeper channel, that is, width to depth ratio decreased.

CONCLUSIONS

The following specific conclusions on the Elk River problem are listed by categories for comparison with conclusions from other Elk River papers.

Aggradation/Degradation

Over the short term some aggradation, particularly in the vicinity of tributary junctions, should be anticipated as tributaries respond to the construction of detention dams. After tributaries have adjusted to altered flow and sediment regimes, degradation along the mainstem Elk River including the Longton reach can be anticipated. Because of geologic controls and the probability of armoring in alluvial reaches, this degradation should not exceed 2 to 3 feet. In fact, geologic controls will limit degradation to alluvial reaches where lowering of bed elevations might be more accurately characterized as "local scour."

Bed Material Size

Over the long term bed material size will increase along the Elk River and in the Longton reach as fines are removed and armoring develops through hydraulic sorting. In terms of altered substrate conditions reaches where a geologic control is covered by a thin veneer of alluvium could be swept clean.

Bank Stability and Width

Two fundamental assumptions of this analysis have been that the banks are composed of predominately cohesive materials and are (and will remain) stabilized by vegetation. Unless a site visit were to contradict these assumptions, channel banks, and width can be considered stable in most reaches. Degradation in alluvial reaches might undercut bankline vegetation and produce local instances of channel widening.

Width to Depth Ratio

With stable banklines and limited degradation, width to depth ratios should change only slightly. With reference to Huang's case studies of response of Kansas streams to impoundment a slight increase in width to depth ratio should be anticipated in some alluvial reaches.

Pool and Riffle Spacing

Redistribution of material produced by short term tributary adjustment to detention structures could alter the composition of bed materials in the riffle and pool sequence as "slugs" of deltaic sediments move along the mainstem. Over the long term under regulated conditions, high flows will be comparatively rare and extended periods of low flow will tend to scour the crossings and fill the pools of the existing meander sequence; however the spacing of the riffle and pool sequence would not be altered by this process.

Sinuosity Change

There is evidence of at least one recent (?) natural (?) cutoff in the Longton reach. A site visit would be required to determine age and origin, but cutoff of a meander loop could produce a radical change in slope, velocity, and transport capacity of the reach. Cutoff of a meander loop is one geomorphic event that could alter the riffle and pool spacing in the reaches above and below the cutoff. Documentation by Stevens (1980) of a flood of record of 200,000 cfs on the Elk River in July 1976 (six times larger than the mean annual flood of 32,800 cfs) indicates a potential mechanism for developing cutoffs of the existing meander pattern.

As a result of the analysis of the Elk River problem and discussion during the Workshop several general conclusions are offered.

Site Visit

A site visit is an absolutely essential element of analysis for any river related problem. Results of any analysis, even at the qualitative level, must be considered tentative until supported by reconnaissance in the field. The time required for a site visit depends on the areal extent and complexity of the watershed. For the Elk River, one day on-site to include examination of the mainstem and several representative tributaries might suffice. Aerial reconnaissance is, in most cases, an essential adjunct to a site visit.

Case Studies

Well documented case studies of river response to similar engineering activity in related physiographic settings constitute an important supplement to analysis of river response. This is particularly true where conclusions or projections of response are required using only a limited data base, as was the case with the Elk River. Hydraulic engineering projects can induce major changes in the hydrologic, hydraulic, and sediment regimes of river systems, and at present, theory alone is not capable of predicting this complex response. Research effort committed to producing documented case studies can provide a valuable resource for evaluating river response.

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PREDICTED RESPONSE OF THE ELK RIVER AT LONGTON, KANSAS

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GENERAL INFORMATION

The Elk River is a tributary of the Verdigris River in the Arkansas River basin in southeastern Kansas. The Elk River reach selected to evaluate is near Longton, Kansas and has a drainage area ranging from 285.7 square miles at the upper end (Section 11-15) to 390.5 square miles at the lower end (Section 3-3). This discussion is an analysis and evaluation of the expected response of this reach of river to the installation of 45 floodwater retarding dams in the upstream reaches of the watershed under the USDA Soil Conservation Service small watershed program authorized under P.L. 83-566.

The watershed consists of gently sloping flood plains and steep bluffs of the Flint Hills escarpment in the upper reaches. In the channel study reach, the area consists of thick beds of sandy shale with interbedded limestone ranging from thin beds to thick ledges. Soils of the watershed are thin over the thick limestone ledges. Floodplain soils are deep and friable and are mainly silty clay loams. Land use over the watershed is about 13% cropland, 82% grassland, 3% woodland, and 2% a mixture of other uses. Of the approximately 7,320 acres of woodlands, about 5,350 acres are on the flood plain, primarily in narrow belts adjacent to the Elk River and its tributaries.

The average annual precipitation for Howard, Kansas, located about 12 miles northwest of Longton near the middle of the watershed, is 35.07 inches. The largest total annual precipitation recorded at Howard is 56.07 inches and the smallest is 18.47 inches. Normally, about 75 percent of the precipitation falls during the growing season, April to October. The Elk River floodplain is flooded frequently--two to three flows a year exceed bankfull capacity. Flooding duration in the study reach is usually 24 to 36 hours. Sediment deposition on the flood plain during flooding causes problems in localized areas.

The average growing season is 185 days. An average year would be frost free from 15 April to 17 October. Daily temperatures average 35°F during January and 80°F during July. Extreme temperatures have been above 115°F and below -20°F.

The proposed project is a system of 45 floodwater retarding dams above the study reach to be installed on the major tributary drainages to the Elk River. The dams are to be earthen with vegetated or rock emergency spillways to provide safe passage of the runoff that exceeds the reservoir detention storage capacity. These spillways are planned so that their chance of operation in any 1 year ranges from 4 to < 1%. The principal spillways for the dams are of reinforced concrete with a crest at the elevation of the 100-year accumulation of sediment and have an uncontrolled release rate of 20 cubic feet per second per square mile of contributing drainage area (csm).

Reservoir detention storage capacity is planned to handle from 3.0 to 5.25 inches of runoff from the contributing drainage area. The storage allocated for sediment accumulation ranges from 0.51 to 1.79 inches from the contributing drainage area. An ungated orifice through the principal spillway is to be provided at the elevation of the 50-year accumulation of sediment. After depletion of this storage, the orifice can be plugged and submerged storage is then available below the principal spillway crest elevation.

The State Geological Survey of Kansas issued a report in July 1958 on the rock formations and mineral and ground water resources for Elk County where the study reach is located. The report states that the flood plain alluvium consists of two strata. The lower stratum is coarse material, predominately chert, limestone, and sandstone gravel and ranges from a fraction of an inch to 10 feet in thickness but generally is about 5 feet thick. Sand is intermingled with the pebbles, some of which are 2 to 3 inches in diameter. These deposits are the better aquifers in the area, even though their yield is not large. Household wells in the area usually yield less than 50 gallons per minute. The lower stratum yields water freely but is not continuous over the entire valley. The upper stratum of alluvium is a deposit of mostly clay and silt which grades downward to more sandy materials.

ANALYSIS

This section describes the concepts and methods used to estimate the future form and substrate of the selected reach (between Sections 3-3 and 11-15). The floodplain along the reach is 6.8 miles long and the stream channel length is 10.2 miles long.

General Geomorphology

To make a prediction of performance, it first is necessary to establish the general state of equilibrium of the stream. This is best done by evaluation of the geomorphic setting and as many quantitative factors as possible that are useful in supporting the identified state of equilibrium.

The general trend is erosional; the channel is gradually cutting into the underlying bedrock. The local irregularities of rock elevation in the bed and of depth of alluvium are due to different degrees of erosion resistance and bedding thickness. Within this reach, the stream flows across the Lawrence Shale Formation, which has a thick limestone stratum and a sandstone member of variable thickness. The sandstone member (Ireland) is a significant aquifer for domestic water wells, even though its yield is relatively low (1 to 10 gpm).

Field inspection, probing, and sampling of the channel bed indicate that, with a few exceptions, bedrock is within 48 inches below the stream bed. This indicates, without doubt, that the stream is in a very active state of degradation, but degradation is restricted by the resistant bed material. With the exception of limestone encountered at the upper end of the reach and sandstone at the lower end, all bedrock in the bed is shale.

Field inspection indicated the streambanks are in alluvium, with two exceptions. One is where the stream encroaches on the valley flank, exposing shale bedrock in the right bank (Section 3-5). The other exception is located in Section 11-12 where the channel is not encroaching on the valley flank. The streambank is in shale that extends 12 inches above the waterline and is capped with alluvium.

The limestone at the upper end of the reach (Sections 13-2 to 11-14) has significantly restricted the valley width compared to the shale upstream and downstream from this location. This restriction is verified by the existence of a limestone ledge outcrop across the channel. Another valley restriction occurs just below Longton near Section 11-1. There is no rock ledge outcrop in this area but limestone talus is identified on the right streambank. In addition, very coarse angular limestone fragments were described in an alternate bar about 1 mile downstream near Section 3-4. The fragments range in size to 36 inches in the longest dimension with the average being 6 to 8 inches. Downstream from the study reach, about 2 miles below Section 3-3 near Oak Valley, there is a third valley restriction, probably the result of sandstone (believed to be the Ireland member). This restriction, along with a significant addition of drainage area between Sections 11-1 and 3-7, needs to be recognized in any prediction about the study reach.

The stream valley form is dominated by the resistant geologic formations and the pattern cannot be predicted in terms of a consistently recurring sinuosity. There are two major patterns of sinuosity established by the channel--one with a wave length of about 2 miles, and the other with a wave length varying from 1,000 to 3,000 feet.

Hydraulics

The data submitted indicate that the stream transports 90% of the sediment as wash load. The project is expected to reduce the sediment discharge by about 60%, but the proportion of wash load to bed load is not expected to change much. The total sediment yield at the lower end of the study reach is estimated to be 31,000 tons per year before project installation and 13,000 tons per year after installation.

Storm runoff values are tabulated below for the bankfull channel condition. This is generally the maximum tractive stress condition and can indicate the time of maximum bed load movement. The existing bankfull discharge in the study reach is about 24 csm.

Table 1. Channel bankfull conditions.

Project Status	Runoff (in.)	Frequency Return Pd. (yr.)
w/o project	0.8	0.6
w/ project	1.6	1.8

Based upon this examination, it can be seen that the "maximum stress" on the bed will be less frequent. Instead of at least annually, it will be only about every other year.

Installation within the watershed of floodwater retarding dams with fixed releases will result in above-average flows for a prolonged period. Table 2 shows the approximate amount and duration of prolonged flow.

Table 2. Approximate amount and duration of prolonged flow.

Section	Drainage Area		Spillway Controlled Release Outflow		Flow Depth (ft)	Bank Full Discharge (cfs)
	Size (mi^2)	Controlled (%)	Rate (cfs)	Duration (hours)		
11-15	284.5	59	3,370	110	13	5-6,000
11-1	297.1	57	3,370	110	9	5-6,000
3-7	341.0	56	3,791	110	10	7-9,000
3-3	390.5	60	4,651	110	10	7-9,000

The prolonged flows will start 18 to 24 hours after the flood crest passes (based upon the synthetic 6-hour storm provided).

Twenty-three of the 45 project dams to be installed are designed to control the 25-year frequency flood event. These dams have overflow emergency spillways which will result in outflow greatly beyond the 20 csm release rate for storms with a magnitude greater than those with a 25-year return period. The runoff for a 6-hour storm of a 25-year return period is 4.5 inches. About 3.5 inches of this amount will be temporarily detained by each retarding structure. Nearly 60% of the drainage area above the study reach is controlled by floodwater retarding structures.

After the project is installed, out-of-bank flow will still occur as a result of the runoff from a 6-hour storm having a 25-year return period. This is based on the data supplied. The unit hydrograph discharge is 14.74 csm per inch of runoff. There is about 4.5 inches of runoff for this size storm. Therefore, the stream discharge will be 14.74 x 4.5, which is about 66 csm and exceeds the bankfull capacity of about 24 csm.

After the project is installed, the frequency of flooding due to out-of-bank flow can be determined by first dividing the 24 csm by 14.74 csm per inch to obtain a storm runoff value of 1.62. By use of the frequency-runoff curve provided, a storm of 6-hour duration and a 1.8-year return period will result in 1.6 inches of runoff.

The frequency of flooding due to out-of-bank flow will be reduced but still will be a common occurrence. However, the duration of flooding will be less. The long duration flows from the principal spillways are about one-half the channel capacity. The added flows from that portion of the watershed not controlled by dams will contribute the remainder of the runoff necessary to cause flooding. This portion of the runoff will be flashy or of short duration.

ADDITIONAL DATA REQUIREMENTS

The review and analysis of information provided indicates that the existing physical characteristics of the channel study reach are dominated by the geologic conditions and not the hydraulic or fluvial forces. The streambed is controlled by bedrock. However, information about the streambanks is very limited and general. To fully verify the streambank stability, it is desirable to collect additional data on the streambank material. These data would allow evaluation of both the structural stability of the streambanks against slides and their resistance to erosion from flow in the channel.

A review of old aerial photos and maps to provide information on the rate of stream alignment changes is advised. Photos taken before and after major storms would be especially valuable.

Examination of other watersheds in the area which have had similar project development would be very valuable in making evaluations. A detailed profile of the channel thalweg would be of value in contrast to the low flow water surface for hydraulic purposes.

It is necessary to examine upstream channel conditions for availability of bed materials to be transported into the reach under study. The channel performs and responds as a system in conjunction with the watershed, and a single reach cannot properly be evaluated without information about the other parts. In this case, the long-term performance of the channel bed materials will be greatly influenced by the bed materials upstream as well as the runoff.

This project is located in a subhumid region and, as a result, vegetation within the channel section will seasonally affect its flow characteristics. Knowing the woody and herbaceous climax species can provide some insight into the potential for an accelerated rate of channel choking, protection of the banks against erosion due to flow impingement, and also the need for maintenance thrugh the removal of snags and windfalls. Some tree species are significantly more susceptible to damage at an early age than others. Some have deeper or more dispersed root structure than others. Each of these items is valuable in anticipating the effects of vegetation.

EVALUATION

On the basis of available data, the channel study reach has been evaluated to anticipate the project's short and long term alterations to

1. Meander pattern,

2. Configuration of the channel,

3. Substrate material, and

4. Pool-riffle sequence.

The meander pattern is very stable. The streambanks are covered with mature vegetation, the channel is very narrow and uniform, and there are no extensive point bars. The hydraulic stress of bankfull flows has been very frequent. On the basis of past demonstrated stability, a reduced frequency of stress will likely result in an even more stable meander pattern in the future, or essentially no change.

The channel cross section shape or configuration also appears to be very stable. In many areas it is made up of a compound slope on each bank. There is a flattened slope in the bottom 2 to 4 feet, and above it is the steepest slope, about 1 to 1, for a height of 6 to 15 feet. Above this steep section there is usually a flattened bench-like slope, topped off with another steep section. The base of the streambank is a gravel layer resting on bedrock of shale, limestone, or sandstone. The generally benched slope and solid foundation provide a generally stable section, as demonstrated by the presence of mature vegetation on the bank.

The Elk River location in a subhumid area is adequate for bank vegetation. The average velocity of streamflow at bankfull capacity ranges from 2.5 to 4 feet per second throughout the reach. The maximum velocity at the point of extreme nonuniformity on curves is greater, but probably no more than 8 feet per second. These relatively low velocities will not inhibit vegetative growth from maturing in the future. The vegetation will provide increased bank protection against hydraulic erosion. The vegetated banks will tend to reduce the velocity at the boundary layer causing some sediment deposition. Channel cross sectional areas are not very likely to diminish as the stream normally carries little sediment and the project should reduce the sediment load in the future by one half.

The substrate material consists of either exposed bedrock or coarse gravel. The change in gradation is not anticipated to be significant.

Because the streambanks in the study reach will be more stable, the dominant source of bed materials will increasingly be the transient bed load moving out of upstream reaches. There should be no long term change in bed material characteristics due to the existing volume of material in the "pipeline." The rate of delivery of bed material will be reduced to about one-half the present rate and, therefore, the volume of bed material should be ample for a very long time since the maximum hydraulic stress at bankfull flow still will move the bed materials but at a less frequent rate.

The bed materials are expected to become generally coarser through removal of some of the sand sizes, because the reservoir release will increase the duration of one-half bankfull flows. This should result in a veneer type of armoring on the bed after each flood event. The veneer size can be evaluated for stability using the Shields diagram. The gravelly bed material is not expected to become choked with silts and clays, since there will be a cleansing at least biannually (on the average) when bankfull flow disturbs the bed surface during its transport.

There will be no significant deepending of the channel since it rests on bedrock of moderate resistance. Bed materials form the pool riffle sequence, but the underlying bedrock restricts the pool depth. In at least one location the riffle is created by a bedrock ledge outcrop crossing the stream.

CONCLUSIONS

The channel study reach of the Elk River demonstrates itself as being very stable, and its characteristics are controlled by geologic factors rather than hydraulic forces. The potential for degradation of the channel bed is limited because of bedrock that is exposed or at shallow depth.

The installation of floodwater retarding dams for controlling runoff from 60% of the watershed will generally result in an even more stable channel. A slight coarsening of the bed material will occur.

The potential for channel choking or obstruction of flow will increase because of increased aging of trees and other vegetation. In addition, growth will provide increased protection against erosion at locations that are lower on the banks and more vulnerable to flow impingement. Both of these effects will result in increased stability of channel alignment.

Most of the answers on anticipated changes would be provided by an examination of the study reach data along with field verification by a person knowledgeable of stream channel behavior and experience with similar streams in the humid or subhumid area. This, along with a simple analysis using the Shields diagram, should provide a good estimate of the substrate size likely to result.

ASSESSMENT OF ANTICIPATED CHANNEL CHANGES IN THE ELK RIVER NEAR LONGTON, KANSAS DUE TO UPSTREAM DEVELOPMENTS

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Consultant, Boulder, Colorado, U.S.A.

INTRODUCTION

The Elk River is a stream draining the east flank of the Flint Hills Escarpment primarily in Elk County in southeastern Kansas (Figure 1). The reach of interest here is at Longton, Kansas which is approximately 70 miles east-southeast of Wichita. At the downstream end of the reach, the drainage area is 405.3 mi^2.

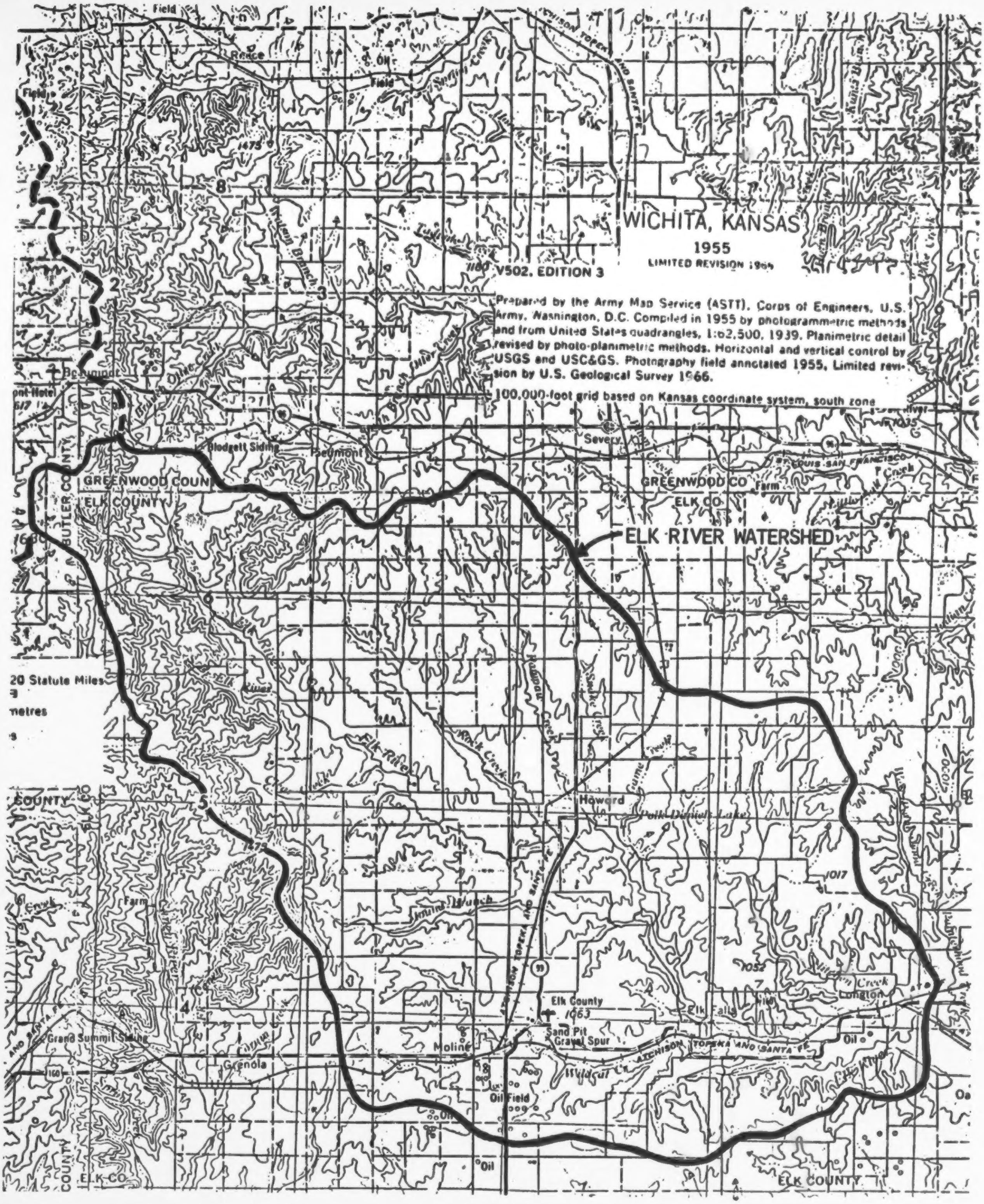
The floodplain of the Elk River is the prime agricultural land in the area. Beef production is the main agricultural activitity. The native pasture, which covers over 76% of the watershed, is utilized as grazing land. Feed grain and alfalfa are produced on the floodplain. Almost 80% of the floodplain is in crops. The 15,375 ac of floodplain cropland represent approximately 45% of the cropland in the watershed so farmers try to keep these lands in production despite frequent damaging floods. On the average, two to three flows a year exceed bankfull capacities. Flooding usually occurs during the growing season.

The U.S. Department of Agriculture, Soil Conservation Service has designed a system of 48 floodwater retarding structures to be installed in the Elk River catchment. The function of these structures is to store floods in the headwaters so as to mitigate flood damage to the cropland on the floodplain downstream.

The system of earth dams will provide 50,253 ac ft of floodwater detention storage and 12,942 ac ft of sediment storage. The system of structures will control the runoff from 239.8 mi^2 which is 59% of the watershed area.

Floodwater storage will be provided for from 3.0 to 5.25 in. of runoff from the upstream drainage area. Also, storage will be provided for the 10-year accumulation of sediment, this volume being equivalent to 0.51 to 1.79 in. of sediment yield from the upstream drainage area. An ungated orifice will be placed at the elevation of the 50-year accumulation of sediment.

The principal spillway of each structure will be reinforced concrete or comparable quality material with a single stage inlet. The uncontrolled design release rate is 20 ft^3/s per mi^2 of drainage per in. of runoff.



Prepared by the Army Map Service (ASTT), Corps of Engineers, U.S. Army, Washington, D.C. Compiled in 1955 by photogrammetric methods and from United States quadrangles, 1:62,500, 1939. Planimetric detail revised by photo-planimetric methods. Horizontal and vertical control by USGS and USC&GS. Photography field annotated 1955, Limited revision by U.S. Geological Survey 1966.

100,000-foot grid based on Kansas coordinate system, south zone

Figure 1. Elk River Watershed upstream from Longton, Kansas

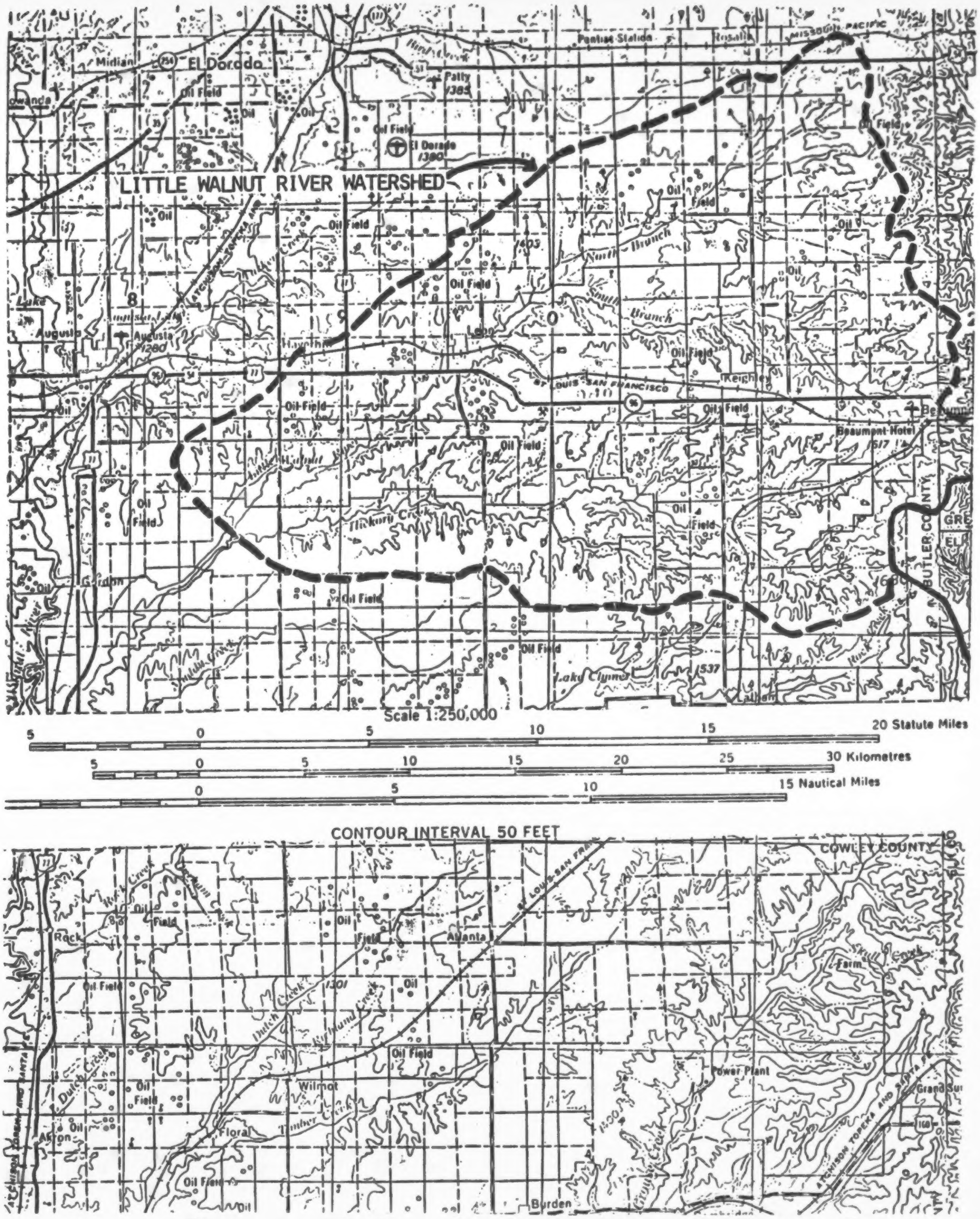


Figure 1. Elk River Watershed upstream from Longton, Kansas (Continued).

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Each dam has a vegetated or rock emergency spillway to discharge runoff exceeding reservoir storage capacity. The chance of the spillway operating in any one year is 4% or less.

The question addressed herein is, "What will happen to the morphology of the Elk River as a result of changes in streamflow and sediment discharge caused by the construction of the 48 small flood retention reservoirs upstream?" Specifically, both the short and long term changes in meander pattern, channel configuration, substrate material, and pool riffle sequence are desired.

GENERAL DESCRIPTION

Climate

The climate of southeastern Kansas is subhumid. The average annual precipitation is 35.07 in. The largest annual precipitation recorded at Howard, Kansas was 56.07 in. in 1961; the smallest was 18.47 in. in 1956. Normally, approximately 75 percent of the precipitation falls during the growing season, April to October (U.S. Department of Agriculture 1967).

The average growing season is 185 days, and in a normal year the area is frost free from 15 April to 17 October.

Temperatures generally average 35°F during January and 80°F during July. Extreme temperatures have been above 115°F and below -20°F.

Valley

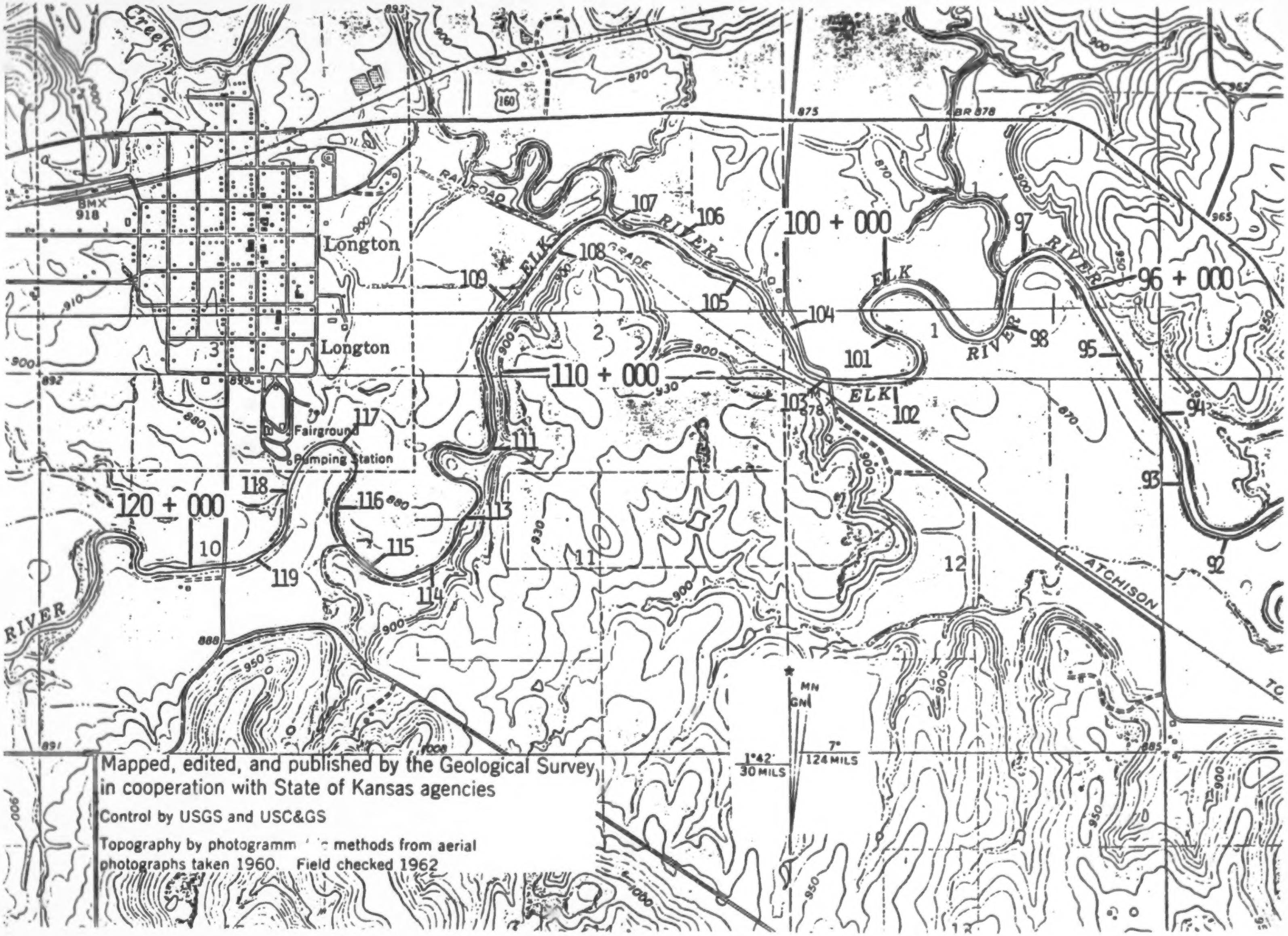
The valley of the Elk River in the vicinity of Longton, Kansas lies in an east-west direction and is relatively straight (Figure 2). The valley floor is approximately 3500 ft wide on the average, and is some 100 to 150 ft below the level of the surrounding hills.

The north side of the valley floor from Longton to 3 mi west of Longton is terraces (Verville et al. 1958). The stream-laid deposits of gravel, sand, silt, and clay are as much as 40 ft thick. The course materials, predominantly chert, limestone, and sandstone gravel are commonly found in the lower zone ranging from a fraction of an inch to 8 ft in thickness. Sand is intermingled with the pebbles, some of which are 2 to 3 in. in diameter. The upper part of the deposit consists mostly of clay and silt but grades downward to more sandy material.

At Longton, the valley slope changes from 6.5 ft/mi upstream to 5.0 ft/mi downstream. The reason could be rock outcrops on the valley floor. Downstream from Longton the valley is wider and there are no terraces, at least to Elk City.

River Morphology

Shown on Figure 2, the study reach is that 50,000 ft long section of the Elk River between River Station 96+000 and 146+000. The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Longton
Longton
BMX 918
160
RAILROAD
GRADE
Fairground
Pumping Station
ELK RIVER
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Mapped, edited, and published by the Geological Survey in cooperation with State of Kansas agencies
Control by USGS and USC&GS
Topography by photogramm[illegible]c methods from aerial photographs taken 1960. Field checked 1962

Figure 2. Elk River near Longton, Kansas

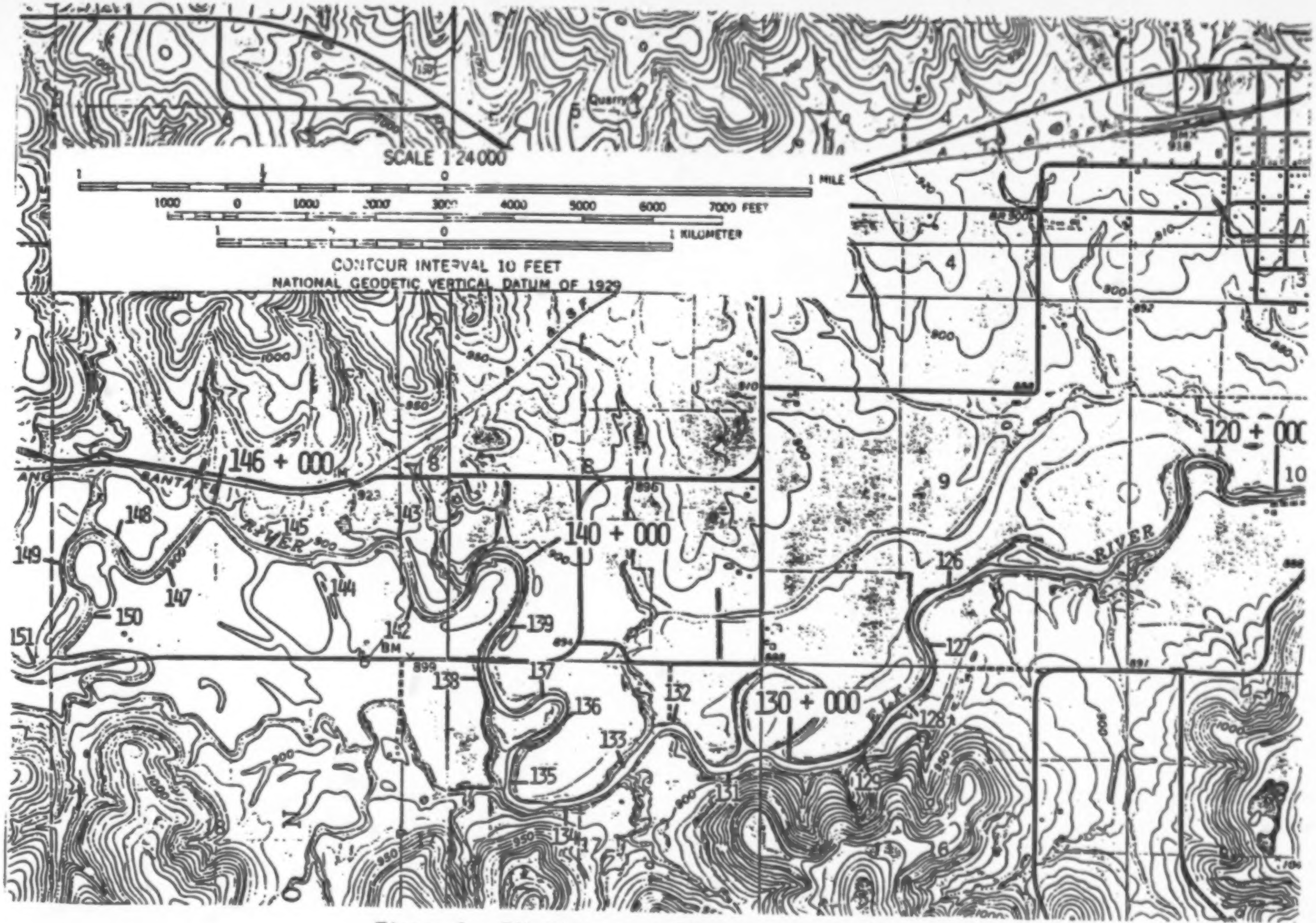

Figure 2. Elk River near Longton, Kansas (Continued).

stationing is in feet. River Station 0+000 was chosen as the U.S. gaging station downstream from Elk City. Below this gage the river cuts through the hills and enters a reservoir.

The slope of the river channel in the study reach changes from approximately 1.5 ft/mi at the downstream end and to approximately 3.5 ft/mi at the upstream end. The channel profile has a discontinuity upstream at the Elk Falls where the river drops at least 10 ft in a 1000 ft long reach.

At the upstream end of the study reach the river is against the north valley wall. Thereafter, the river crosses the valley floor to wander along the south valley wall. Downstream from Longton, the river meanders abruptly to the north side of the valley. In the 5.3 mi length of the valley, the river channel is against the valley wall for approximately 2.5 mi.

In this reach, the sinuosity of the Elk River is approximately 1.8 overall. One cutoff has occurred recently leaving a timbered loop on the floodplain at River Station 130+500.

The bankfull width of the river channel averages 200 ft with variations of 50 ft in either direction. The banks are timbered throughout the entire reach.

The bankfull depth varies from approximately 25 to 50 ft; the average is 33 ft. Thus, the width-to-depth ratio for bankfull flow in the Elk River is approximately 6.0 in the study reach.

Based on the plan, profile, and cross-sectional surveys conducted for the Elk River Watershed, the bankfull discharge for the reach is in the range from 5000 ft^3/s to 10,800 ft^3/s; the average is 7800 ft^3/s.

Streamflow

Streamflow records have been kept for the U.S. Geological Survey gaging station on the Elk River at Elk Falls from January 1967 to the current year. The drainage area for the station is 220 mi^2.

In the 11 years of complete annual record, the average discharge has been 181 ft^3/s which corresponds to an average runoff of 11.18 in./year from the entire catchment upstream.

Monthly records of streamflow have been compiled for the 1968 to 1976 water years inclusive. These are summarized in Table 1.

Table 1. Mean Monthly Streamflow Elk River at Elk Falls (Units of ft^3/s and %).

Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
153	233	139	141	135	336	284	250	236	292	24.4	56.7	190
6.83	10.05	6.21	6.28	5.50	14.99	12.26	11.16	10.16	13.02	1.09	2.45	100

In a normal year, streamflow is a maximum in March when 15% of the annual runoff occurs on the average. August is the driest month with only 1% of the annual runoff.

A listing of all floods greater than 4000 ft^3/s at Elk Falls is given in Table 2. Based on the 11 years of annual peaks, the mean annual flood is 32,800 ft^3/s; the coefficient of variation is a rather large value of 1.7.

The flood of record occurred on 3 July 1976 when the momentary discharge reached an estimated 200,000 ft^3/s, a value 6 times greater than the mean annual flood and 40 times greater than the minimum bankfull discharge.

In the 11 years of record, the flood peak at Elk Falls has exceeded the lowest bankfull discharge (5000 ft^3/s) in the study reach 40 times. Eighteen of these 40 floods occurred in the growing season.

Sediment Yield

There has been only one suspended sediment sample taken in the Elk River in the period of record. Sediment yield has been estimated by the Soil Conservation Service in order to design the detention reservoirs. Their estimates of sediment yield range from 0.51 in. to 1.79 in. equivalent erosion in 100 years.

The development plan is to store 12,942 ac ft of sediment from a drainage area of 239.8 mi^2 in 100 years. If it is assumed that all sediment is stored, the equivalent sediment yield is approximately 0.01 in./year. This value corresponds to an average annual suspended sediment concentration of approximately 900 mg/l.

By way of comparison, the rate of sedimentation in Howard Lake near Howard, Kansas (Figure 1) since 1936 is equivalent to an erosion rate of 0.007 in./year.

The Soil Conservation Service (1967) reports that upland erosion is a serious problem. Floods occurring in the springtime after the thaw but before vegetative cover develops cause extreme land damage. Furthermore, the sediment deposited on the floodplain is infertile silt, sandy

Table 2. Floods in the Elk River at Elk Falls.

Water Year	Date	Hour	Discharge ft³/s
1967	5 Jul		7,240
1968	7 Oct	1900	5,850
	3 Apr	1300	4,150
	25 May	1045	9,610*
1969	9 Oct	1300	5,480
	2 Nov	2400	7,900
	24 Mar	0330	5,480
	18 Apr	0515	10,100
	27 Apr	0500	10,100
	8 May	1400	4,720
	30 May	1300	8,940
	1 Jun	1300	19,100*
	24 Jun	0600	4,860
	16 Sep	0800	5,160
1970	12 Oct	2100	9,460
	1 Apr	1715	12,400
	18 Apr	2015	29,300*
	12 Jun	0600	6,380
1971	3 Jan	1645	4,580*
1972	15 Dec	1045	6,220
	18 Jul	2200	15,200*
1973	13 Nov	0915	7,050
	30 Dec	0915	8,430
	1 Feb	1330	4,460
	4 Mar	1630	7,110
	6 Mar	2000	6,170
	9 Mar	0415	9,620
	10 Mar	2130	10,700*
	25 Mar	1045	6,420
	31 Mar	0700	5,790
	15 Apr	2100	4,230
1974	11 Oct	1630	5,100
	20 Nov	1230	5,580
	4 Dec	1115	8,920
	9 Mar	0400	4,400
	10 Mar	0745	14,500*
	30 Apr	1100	4,290
	22 May	0915	6,460
	24 May	0015	5,160

Table 2. (Concluded)

Water Year	Date	Hour	Discharge ft^3/s
1975	31 Oct	0400	12,900
	3 Nov	1000	22,400*
	31 Jan	0100	6,830
	27 Mar	1745	6,570
1976	28 Apr	1500	8,120
	3 Jul	0700	200,000*
1977	13 Apr	2130	6,420
	21 May	1345	9,100
	22 Jun	1130	24,100*
1978	24 Mar	1230	4,370
	19 May	1415	11,700*

[a] The symbol "*" denotes the peak discharge for the year.

silts, and clays. From the information given by the Soil Conservation Service (1967), Verville et al. (1958), and Bell and Rowland (undated), it is concluded that the sediment yield of the Elk River catchment must be mostly silt and clay.

Bed Material

The material on the bed of the Elk River in the study reach was investigated in February 1980. The data collected indicate that the riverbed level is controlled by rock outcrops and that there is but a thin vein of coarse alluvium on the bed at other locations.

Immediately above the upstream end of the reach (RS 152+000) the river is flowing on limestone. Between here and the abandoned railroad grade (RS 108+000) the riverbed consists of Lawrence Shale overlain with alluvium and an armor layer of limestone, sandstone, and shale material in the gravel, cobble, and boulder sizes. The particles are very angular indicating they have not travelled far from their parent rocks. In the areas sampled, the depth of alluvium varied from zero (RS 138+000) to 4 ft (RS 124+000).

Downstream from the abandoned railroad grade, the river flows on a relatively thin layer of alluvium on top of the Ireland Sandstone member of the Lawrence Shale formation. At River Station 103+000, sandstone was encountered 6 in. below the bottom of the channel. At other locations an armor layer of gravel and boulder sandstone particles covered the bed.

It is apparent that the bed of the Elk River is not, in general, alluvial. The bed level is controlled by outcrops of rock or gravel and boulder particles of angular rock obtained from the outcrops but not transported far from their source. It is inferred that the bedload of this river is very small.

Bank Material

The river banks are alluvial at almost all cross sections investigated in February 1980. One exception is at River Station 103+000 where the river is against the south valley wall. Here the right bank is limestone talus of gravel and cobble sizes.

The upper (above the low-water channel) slopes of the river banks are approximately 2.5 horizontal to 1 vertical and support a growth of old timber, and grass. In locations where the upper slopes are steeper but not yet caving, only grass and willows grow. It appears that the alluvial banks are composed almost entirely of silt and clay materials.

Bank erosion was observed in February 1980 at two locations on the outside of bends. Here the caving banks were vertical and bare of vegetation.

The river banks are timbered all along the reach and, at many cross sections, there appear to be natural levees on the floodplain side, in some cases, at least 3 feet high. There are no manmade levees.

Floodplain

The valley floor of the Elk River in the study reach is on the average 3500 ft wide. The land which is floodplain is cropped with alfalfa and feed grains. There are roads and small drainage channels on the floodplain. The cross-sectional surveys indicate the surface of the floodplain is very irregular.

EFFECTS OF DEVELOPMENT

The 48 flood retention reservoirs in the upstream catchment will affect the amount and delivery of water and sediment to the Elk River. Estimates of these have been made by the Soil Conservation Service.

Water Yield

The annual water yield from the Elk River catchment will be only slightly decreased because of evaporation from the water ponded in the flood retention reservoirs.

Flood Hydrographs

Floods will be decreased greatly when the 48 flood retention reservoirs are constructed. The peak of the unit hydrograph will be reduced from 30.3 to 14.7 ft^3/s per mi^2 per in. of runoff. That is a reduction of approximately 50%. Also, the volume of water in the main part of the hydrograph (first 36 hours) will be reduced approximately the same amount.

Sediment Discharge

The sediment trapped in the flood retention reservoirs will result in an estimated 58% reduction in the annual sediment load in the Elk River at Longton. As the bed-load transport is very small now, the reduction will be primarily in the suspended load which is presumed to be mostly silt and clay.

RESPONSE OF THE RIVER

Methods

The response of the Elk River to the great decreases in flood peaks and sediment discharge can be predicted on the basis of general geomorphic relations developed from experiences in many rivers in many parts of the world. Normally, I would supplement the geomorphic analysis with some mathematical modeling of the water and sediment transport, and changes in bed-material size and configuration. In this case, the mathematical modeling is foregone for these reasons: 1) the river bed is not entirely alluvial which means that the bed load at a section may not be related to the local shear stress but controlled entirely by the amount coming in from above; 2) much of the material on the bed is very angular gravel, cobbles, and boulders which have not been transported far from their parent rock. Existing bed-load equations are for more rounded rock and not this talus material; 3) the river meanders appreciably, so average flow properties at any cross section may not represent values to be used in existing gravel transport equations (which were developed for more regular reaches of rivers); 4) it appears that the amount of bed load in the river is very small and of little consequence; and 5) there are no field data with which to calibrate a mathematical model of the sediment transport process.

The geomorphic relations employed in this paper are those developed by Schumm (1977) but modified slightly based on experience on tropical islands in the Orient where sediment concentrations reach values as large as 100,000 mg/l in the rivers and 20,000 mg/l in the irrigation canals.

Geomorphic Equations

The basic premise is that the river morphology is in "regime." That is, the width, depth, and other features have adjusted over a long period of time to conform to the stresses caused by the water and sediment load. It appears that the Elk River is in regime because there are no reports of geomorphic change caused by the enormous flood of 3 July 1976. Also, if a flood with a peak six times the mean annual flood could not cause widespread bank caving and channel change, one must presume the banks and bed are stable and will not respond quickly to changes in water and sediment discharge.

Schumm's expression relating river channel morphology to water discharge is

$$Q \sim \frac{b,d,\lambda}{s} \tag{1}$$

in which, Q = either the mean annual discharge, or the mean annual flood

b = bankfull width

d = bankfull depth

λ = meander wave length

s = riverbed slope

Equation 1 is not an equality but merely a shorthand method of saying that the magnitudes of the width, depth, and wave length are directly proportional to the magnitude of the streamflow and the bed-slope is inversely proportional to the magnitude of the streamflow.

In relating channel morphology to sediment load, Schumm assumed that the percent of silt and clay in the wetted perimeter is inversely proportional to the bed load and that the total load is directly proportional to the bed load. In general, these assmptions may be valid. However, it would seem that one can expand Schumm's analysis to reflect the fact that some rivers have noncohesive beds but transport mostly silt and clay. These rivers can respond differently depending on whether it is the silt and clay load or the noncohesive load which is affected by development.

Schumm has shown that channel width and depth are related closely to the percentage of silt and clay (M) in the sediments forming the perimeter of the channel.

For the range of channels studied by Schumm

$$\frac{b}{d} = \frac{225}{M^{1.08}} \tag{2}$$

$$b = 37\,\frac{Q_{0.38}}{M^{0.39}} \tag{3}$$

and

$$d = 0.6\,M^{0.34}\,Q^{0.29} \tag{4}$$

Here Q is the mean annual discharge in ft³/s and b and d have units of ft.

In general, it is expected that the values of the coefficients and exponents in the above equations vary somewhat with geological setting and size of river.

Equations 2, 3, and 4 indicate that, for these particular channels, the percent silt and clay was inversely proportional to the river size or

$$M = \frac{57}{Q^{0.26}} \tag{5}$$

One can argue that it is the amount of silt and clay in the banks only that determines the width of the alluvial channel and that the composition of the bed is less important in determining the width. That is, one can have the same channel width and shape with a sand bed or gravel bed provided the banks are clay. It is the cohesion in the clay that allows clay-bank channels to withstand higher stresses developed in narrow channels. Thus, one can use the wash load Q_w (silt and clay transported in suspension) as the indicator of the effect of sediment on channel width. Furthermore, M is directly proportional to Q_w so Equation 3 can be expressed as

$$b \sim \frac{Q}{Q_w} \tag{6}$$

The same type of argument applies to depth. If the wash load is large (large concentration of silts and clays), the river channel is narrow, flood depths are large, a large amount of sediment can be deposited on the flood plain, thus building up high banks. Therefore Equation 4 can be expressed as

$$d \sim Q_w, Q \tag{7}$$

It follows that the bankfull width-to-depth ratio b/d is almost entirely dependent on the wash load.

Following this type of reasoning and by considering Q constant, an expression relating channel morphology to wash load (silt and clay carried as suspended load) can be derived

$$Q_w \sim \frac{d,P}{b,\lambda,S} \tag{8}$$

in which P = sinuosity of the channel

λ = meander wavelength

S = riverbed slope

It has been assumed that the valley slope is an independent variable, the valley having been carved by hydrologic events no longer directly influencing the river channel shape.

Equation 8 is equivalent to Schumm's if one assumes Q_w is inversely proportional to the bed-material load Q_b. In this expression, the bed slope change is accomplished by changes in sinuosity and meander wavelength and not by aggradation and degradation.

Now, it is known that for rivers with alluvial beds and fixed cross-sectional shape,

$$Q_b \sim \frac{Q,S}{d_{50}} \tag{9}$$

This is Lanes' (1955) qualitative relation between bed-material load Q_b, water discharge Q, median bed sediment d_{50}, and riverbed slope S. The expression can also be derived mathematically by relating shear stress on the bed with the transport of noncohesive bed particles. The change in slope in Equation 9 is accomplished by aggradation or degradation and not by a change in alignment as in Equation 8.

For a fixed discharge, then

$$Q_b \sim \frac{S}{d_{50}} \tag{10}$$

Now, Equations 1, 8, and 10 form a set of relations among water and sediment discharge and alluvial river morphology. The assumptions are that the wash load and water discharge have the major influence on the cross-sectional shape, slope, and sinuosity of the channel, and the bed-material load relates closely to only the material size d_{50} and the slope S.

For the relatively straight forward cases of an increase or decrease in discharge, washload, or bed-material load alone, the response of a channel to change is

$$Q^{+} \sim b^{+}, d^{+}, \lambda^{+}, S^{-} \tag{11}$$

$$Q^{-} \sim b^{-}, d^{-}, \lambda^{-}, S^{+} \tag{12}$$

$$Q_w^{+} \sim b^{-}, d^{+}, \lambda^{-}, S^{-}, P^{+} \tag{13}$$

$$Q_w^{-} \sim b^{+}, d^{-}, \lambda^{+}, S^{+}, P^{-} \tag{14}$$

$$Q_b^{+} \sim S^{+}, d_{50}^{-} \tag{15}$$

$$Q_b^{-} \sim S^{-}, d_{50}^{+} \tag{16}$$

Here, a plus or minus exponent is used to indicate how, with an increase or decrease of water or sediment discharge, the various aspects of channel morphology change. The plus exponent indicates an increase and a negative exponent indicates a decrease. No change is denoted with a zero exponent.

One physical interpretation of the set of expressions given above is this. The water discharge represents processes which tend to erode the banks and bed and straighten the channel alignment. The wash load represents processes which tend to build and maintain banks and to contort the alignment. The bed-material load represents processes which tend to change mainly the level and configuration of the bed and the size of material on the bed. The physical interpretation is important because the expressions may not represent all possible sequences.

Immediate Response

The immediate effect of decreasing the flood discharges and sediment load in the Elk River system should be as follows.

1. The bed material will be coarser as the supply of bed-material load from upstream is decreased. There will be less fines on the river bed, however, little or no decrease in riverbed

level in the study reach is anticipated as the armor coat is nonalluvial, angular, and very course in most places. Rock outcrops in the bed at other places. The riverbed slope will not decrease due to degradation. Therefore

$$Q_b \sim \frac{S^o}{d_{50}^+} \qquad (17)$$

It is assumed that the tributary channels on which the dams will be constructed have essentially coarse gravel, nonalluvial beds. If the beds of some tributaries are composed of sand, these channels could supply an excess bed load to the main stream during the first few years of operation.

2. The decrease in flood discharge will decrease the processes tending to widen the channel by erosion and undercutting the banks. As the bed level will not change appreciably and sediment deposition on the floodplain will decrease, the bank full depth should not change immediately. The meander wave length will not change rapidly as considerable time (a century or more) is required for the river to work laterally across the floodplain destroying one pattern and building another if the existing floodplain deposits are tough cohesive materials. Then it follows that the channel should narrow if the sediment required to build banks is available. That is

$$Q^- \sim \frac{b^-,\ d^o,\ \lambda^o}{S^o} \qquad (18)$$

A new and more vigorous growth of vegetation may result on the banks as plant scouring will be reduced because flow velocities will be lower.

3. There will be a considerable decrease in the amount of wash load carried by the river. The turbidity of the water will be much less. Wash load is the material from which new banks are made. New deposits on the banks could be facilitated by more luxurious vegetation on the banks. It follows then, from the geomorphic expression relating wash load and channel shape, that the channel should widen initially. That is

$$Q_w^- \sim \frac{d^o,\ P^o}{b^+,\ \lambda^o,\ S^o} \tag{19}$$

This is in contradiction with the conclusion drawn from the geomorphic expression relating discharge and channel shape. However, the influence of Q_w on the width-to-depth ratio is much more pronounced than the influence of the water discharge. Therefore, the channel should tend to widen.

It is concluded that the short term response will be a channel which is deeper than required and which is slightly too narrow now. The channel will not degrade but the bed will become cleaner as the finer material is removed leaving even more armor coat than is currently on the bed. The pool and ripple sequence will remain unchanged.

Long Term Response

If left unimpeded over a very long period of time, the Elk River would move its channel laterally across the floodplain eroding one bank and building the other. Even though the farmers would not tolerate the river destroying their cropland by moving laterally, it is useful to estimate the change in river form which would result from migration. Because of the changes in water and sediment discharges, the new river will have a different form than the current river. The differences can be estimated as follows.

1. The long term response to the decrease in bed-material load will be the same as the short term response. That is, the river will not be able to degrade because the bed material is very coarse and there are rock outcrops controlling the bed level. Therefore, the slope will not change due to degradation, but the bed material will become coarser. That is, the new regime will be

$$Q_b^- \sim \frac{S^o}{d_{50}^+} \tag{20}$$

2. The long term response to the decrease in water discharge and wash load can be determined by noting that the combined response is

$$Q^{-}\ Q_w^{-} \sim \frac{\overset{+}{b^{-}},\ d^{-},\ P^{-},\ \overset{+}{\lambda^{-}}}{S^{+}} \qquad (21)$$

That is, the new river will be shallower (flood discharges are not as large so the floodplain will not build up as high as before), less sinuous, and will have larger bed slope (due to less meandering, not aggradation). There should be fewer pools and ripples. There are questions concerning whether the channel will become wider (b^{+}) or narrower (b^{-}) and whether the meander wave length will be longer (λ^{+}) or shorter (λ^{-}).

First the width. It is my opinion that the vegetation on the banks will be the added stabilizing factor which will cause the new river to be narrower. Vegetation is effective in slowing down the velocity and trapping fine sediment on the bank.

The meander wave length must increase because the sinuosity will decrease and the slope will increase (due to strengthening, not aggradation).

In order to predict the magnitude of the anticipated long term changes in river form, the geomorphoric equations developed by Schumm (1977) are used here. These equations were developed from rivers with width-to-depth ratios from 2 to 300 so represent a wide range of river forms. The equations are

$$\frac{b_f}{b_o} = {\frac{Q_f}{Q_o}}^{0.38} {\frac{Q_{wo}}{Q_{wf}}}^{0.39} \qquad (22)$$

$$\frac{d_f}{d_o} = {\frac{Q_{wf}}{Q_{wo}}}^{0.34} {\frac{Q_f}{Q_o}}^{0.29} \qquad (23)$$

$$\frac{P_f}{P_o} = {\frac{Q_{wf}}{Q_{wo}}}^{0.25} \qquad (24)$$

and

$$\frac{\lambda_f}{\lambda_o} = {\frac{Q_f}{Q_o}}^{0.48} {\frac{Q_{wo}}{Q_{wf}}}^{0.74} \qquad (25)$$

Here the subscripts "o" and "f" refer to conditions before and after development, respectively.

Because the valley slope remains changed, it follows that

$$\frac{S_f}{S_o} = \frac{P_o}{P_f} \qquad (26)$$

Using the values,

$$b_o = 200 \text{ ft}$$

$$d_o = 33 \text{ ft}$$

$$P_o = 1.8$$

$$\frac{Q_f}{Q_o} = 0.50$$

$$\frac{Q_{wf}}{Q_{wo}} = 0.58$$

the following estimates of the changed river form are obtained.

b_f = 190 ft (5% smaller)

d_o = 22 ft (32% smaller)

P_f = 1.6 (13% smaller)

Currently, there are 17 meanders with a wide range of wave lengths in the study reach. As wave length is inversely proportional to number of meanders, one anticipates there will be one less meander after development. Similarly there should be a reduction of approximately 10% in the number of pools and ripples.

The rate at which the Elk River could move laterally across its valley will be much slower after development. Since there is very little evidence of bank caving now, it is anticipated that it will be decades before one will notice any response. The geological survey indicates that the alluvium on the valley floor is primarily silt and clay but if the river was to encounter noncohesive materials in the former alluvial deposits, response would be much faster. If any local area becomes unstable, for example, if the river cuts into a sandy deposit, this area will probably be stabilized immediately at the request of the local land owners.

Other Considerations

Changes in river form due to changes in wash load and vegetal factors are not so well documented in geomorphic literature as the response due to changes in discharge and bed-material load. Therefore, some caution is warranted. Two studies should be done to improve estimates made above. They are to

1. Develop geomorphic equations based on the data of rivers in southeastern Kansas, and
2. Study the effect 93 flood retention reservoirs in the Little Walnut River catchment have had on the Walnut River. This catchment is immediately west of the Elk River catchment (Figure 1). Also, the vegetation along the river banks must be maintained. There is a question concerning what will happen when the large trees die of old age. A study of the vegetation succession for this river is warranted.

SUMMARY

The anticipated response of the Elk River near Longton to the construction of 48 flood retention reservoirs in the upstream catchment is as follows.

1. In the short term, the material on the riverbed will become coarser.
2. If left unimpeded in the long term, the river would become slightly narrower, much shallower, slightly less sinuous, slightly steeper (but with no aggradation) and the number of ripples and pools would decrease slightly. However, it is anticipated that the land owners will not tolerate lateral migration of the river. Therefore, in the long term, the Elk River will become slightly narrower and will be left with its large remnant bankfull depth developed during a period before dams when water and sediment discharges were much greater.
3. Very little effort will be required to keep the Elk River in its stable regime even after development...if the vegetation on the banks can be maintained. More effort should be given to vegetation studies along this river and land owners should be informed of the importance of river bank vegetation.

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APPENDIX A

CHANNEL CHANGE WORKSHOP: PROBLEM NUMBER 1
YAMPA RIVER, COLORADO

THE QUESTION

The question relative to the Yampa River is "What will happen to the morphology of the stream channel as a result of the changes in stream flows and sediment discharge caused by the construction of a number of reservoirs upstream of a reach of stream?" Specifically, for both a short and a long time after construction of the reservoirs:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material?
4. What will be the pool riffle sequence?

The Yampa River is located in Northwestern Colorado. All the reaches of interest are in the lower part of the river. Data are supplied for four reaches but only two are to be considered in any detail. These two are

1. The Box Elder Reach, and
2. The Lily Park Reach.

Lily Park Reach is located just above the Little Snake River and the Box Elder Reach is located just above the junction of the Yampa River with the Green River. Data were obtained from the U.S. Geological Survey and from the U.S. Fish and Wildlife Service. The full problem set is available from the Instream Flow Group, U.S. Fish and Wildlife Service.

CONTENTS[1]

General location of Yampa River Basin, Colorado

Monthly Streamflow Data for Yampa River Basin, Colorado

Annual Peak Flow Frequency Analysis

Average Monthly Discharge for Yampa River Basin, Colorado

Average Daily Discharge for Yampa River Basin, Colorado

Yampa River Cross Section

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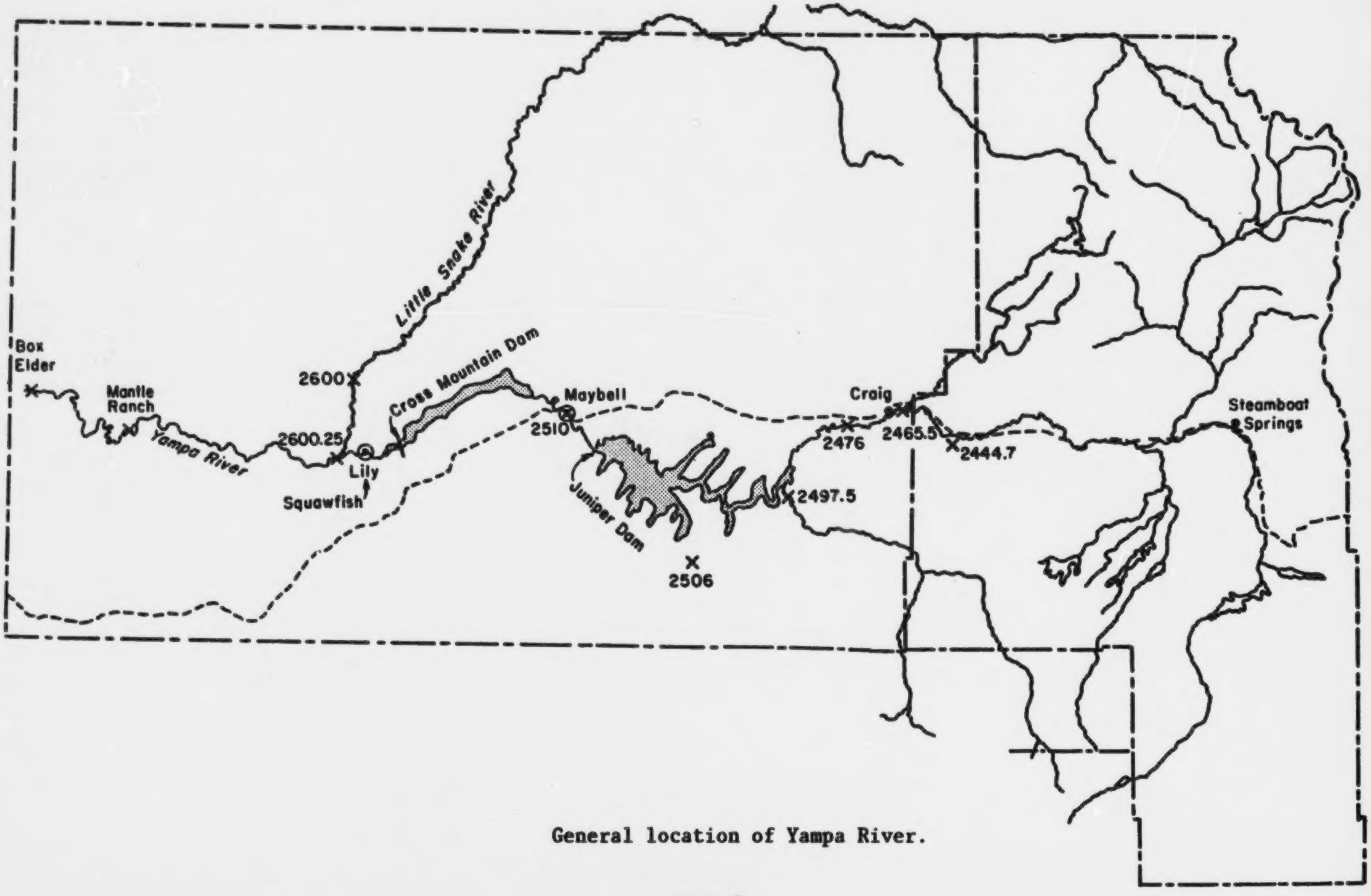
[1] Only selected data are provided in this appendix. Contact the Instream Flow Group of the Fish and Wildlife Service for the complete data set.



General location of Yampa River.

YAMPA RIVER BELOW PROPOSED JUNIPER RESERVOIR
SIMULATION OF MONTHLY FLOWS
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1927	0.	0.	0.	0.	0.	628.	200.	200.	200.	200.	200.	0.	137.
1928	0.	0.	0.	272.	401.	200.	200.	200.	200.	200.	200.	200.	171.
1929	0.	0.	0.	465.	486.	0.	200.	200.	200.	200.	200.	200.	177.
1930	200.	200.	218.	499.	532.	399.	200.	200.	200.	200.	200.	0.	253.
1931	0.	0.	353.	511.	496.	0.	200.	200.	200.	0.	0.	0.	162.
1932	0.	0.	0.	0.	0.	0.	200.	200.	200.	200.	200.	0.	84.
1933	0.	0.	0.	0.	0.	485.	200.	200.	200.	200.	0.	0.	108.
1934	0.	0.	0.	0.	0.	0.	411.	200.	0.	0.	0.	0.	51.
1935	0.	0.	0.	0.	0.	0.	508.	200.	200.	0.	0.	0.	75.
1936	0.	0.	0.	0.	0.	0.	200.	200.	200.	200.	0.	0.	67.
1937	0.	0.	0.	0.	0.	0.	386.	200.	200.	0.	0.	0.	65.
1938	0.	0.	0.	0.	0.	551.	200.	200.	200.	200.	0.	0.	114.
1939	0.	0.	0.	0.	0.	482.	200.	200.	200.	0.	0.	0.	91.
1940	0.	0.	0.	0.	0.	0.	292.	200.	200.	0.	0.	0.	57.
1941	0.	0.	0.	0.	0.	0.	404.	200.	200.	0.	0.	0.	67.
1942	0.	0.	647.	0.	0.	477.	200.	200.	200.	200.	0.	0.	162.
1943	0.	0.	0.	0.	0.	0.	200.	200.	200.	0.	0.	0.	50.
1944	0.	0.	0.	0.	0.	0.	489.	200.	200.	0.	0.	0.	74.
1945	0.	0.	0.	0.	0.	0.	383.	200.	200.	200.	0.	0.	82.
1946	0.	0.	0.	0.	0.	541.	200.	200.	200.	0.	0.	0.	96.
1947	0.	0.	0.	0.	0.	481.	200.	200.	200.	200.	0.	0.	108.
1948	0.	0.	0.	774.	698.	646.	200.	200.	200.	200.	0.	0.	241.
1949	0.	0.	0.	0.	0.	0.	200.	200.	200.	200.	200.	0.	84.
1950	0.	0.	0.	728.	0.	0.	310.	200.	200.	0.	0.	0.	121.
1951	0.	0.	0.	0.	0.	609.	206.	200.	200.	200.	0.	0.	119.
1952	0.	0.	0.	0.	750.	0.	200.	200.	200.	200.	200.	200.	158.
1953	0.	0.	0.	0.	0.	0.	480.	200.	200.	0.	0.	0.	73.
1954	0.	0.	0.	0.	0.	0.	390.	200.	0.	0.	0.	0.	49.
1955	0.	0.	0.	748.	0.	0.	344.	200.	200.	0.	0.	0.	125.
1956	0.	0.	0.	0.	0.	0.	200.	200.	200.	200.	0.	0.	67.
1957	247.	0.	0.	0.	0.	0.	324.	200.	200.	200.	200.	200.	131.
1958	200.	200.	200.	200.	229.	301.	234.	200.	200.	200.	200.	200.	214.
1959	0.	200.	0.	659.	0.	0.	424.	200.	200.	0.	0.	0.	141.
1960	0.	672.	0.	762.	0.	0.	200.	200.	200.	0.	0.	0.	170.
1961	0.	0.	0.	0.	0.	0.	571.	200.	200.	0.	0.	0.	80.
1962	502.	0.	679.	0.	475.	440.	200.	200.	200.	200.	200.	0.	258.
1963	0.	0.	0.	0.	0.	621.	277.	200.	200.	0.	0.	0.	109.
1964	0.	0.	0.	0.	0.	0.	470.	200.	200.	0.	0.	0.	72.
1965	0.	0.	0.	0.	0.	656.	0.	200.	200.	200.	200.	0.	123.
1966	200.	0.	200.	351.	0.	474.	200.	200.	200.	0.	0.	0.	154.
1967	0.	0.	0.	0.	0.	0.	413.	200.	200.	0.	0.	0.	67.
1968	0.	0.	0.	0.	703.	0.	377.	200.	200.	200.	0.	0.	136.
1969	0.	0.	0.	0.	0.	0.	200.	200.	200.	200.	0.	0.	67.

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YAMPA RIVER BELOW PROPOSED JUNIPER RESERVOIR (CONTINUED)

1970	0.	0.	640.	0.	0.	609.	263.	200.	200.	200.	200.	200.	211.
1971	0.	200.	200.	295.	426.	247.	200.	200.	200.	200.	200.	200.	213.
1972	200.	198.	0.	200.	200.	200.	200.	200.	200.	0.	0.	0.	132.
1973	0.	0.	0.	0.	730.	0.	368.	200.	200.	200.	200.	0.	154.
1974	0.	0.	0.	710.	0.	0.	200.	200.	200.	200.	200.	0.	144.
1975	0.	0.	0.	0.	0.	0.	410.	200.	200.	200.	0.	0.	84.
1976	0.	0.	0.	0.	0.	0.	422.	200.	200.	0.	0.	0.	68.
AVERAGE	31.	33.	63.	143.	123.	181.	283.	200.	192.	108.	60.	28.	120.
Q/QANN	2.186	2.281	4.428	10.126	7.880	12.769	19.336	14.114	13.113	7.622	4.234	1.912	100.000
COV VAR	3.006	3.299	2.679	1.300	1.934	1.382	.413	0.000	.206	.932	1.543	2.504	.461
SKEW	3.489	4.476	2.916	1.558	1.679	.838	.522	1	-4.841	-.166	.900	2.140	.840
MAXIMUM	502.	672.	679.	774.	750.	656.	571.	200.	200.	200.	200.	200.	258.
MINIMUM	0.	0.	0.	0.	0.	0.	0.	200.	0.	0.	0.	0.	49.

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STATISTICAL PARAMETERS FOR STATION   CP18        YAMPA RIVER BELOW PROPOSED JUNIPER RESERVOIR
  SIMULATION OF MONTHLY FLOWS


                          ARTH.                    LOG PARAMETERS
              MONTH      AVERAGE     MEAN    VARIANCE  STD. DEV.     SKEW


     1        OCT         30.98    -2.3540    3.1249    1.7677     2.4158
     2        NOV         33.40    -2.3534    3.1329    1.7700     2.4192
     3        DEC         62.74    -2.1152    4.2026    2.0500     1.9218
     4        JAN        143.48    -1.4139    6.6131    2.5716     1.0169
     5        FEB        122.52    -1.6379    6.0033    2.4502     1.2602
     6        MAR        180.94     -.8523    7.6895    2.7730      .5140
     7        APR        283.12     2.3237     .6137     .7834    -6.6479
     8        MAY        200.00     2.3010     .0000     .0000     1.0312
     9        JUNE       192.00     2.0890    1.1011    1.0493    -4.8413
    10        JULY       108.00     -.1374    7.1227    2.6688     -.1655
    11        AUG         60.00    -1.4097    6.0216    2.4539      .9001
    12        SEPT        28.00    -2.2579    3.4524    1.8581     2.1397
    13        ANNUAL     120.27     2.0363     .0389     .1972      .1026


   SAMPLE SIZE   50 YEARS



LOG NORMAL DISTRIBUTION STATION   CP18        YAMPA RIVER BELOW PROPOSED JUNIPER RESERVOIR
  SIMULATION OF MONTHLY FLOWS


                         YEARS FLOW IS NOT EXCEEDED

 MONTH   9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20              Q2 - Q10

 OCT          1.        0.        0.        0.        0.        0.                    0.
 NOV          1.        0.        0.        0.        0.        0.                    0.
 DEC          3.        0.        0.        0.        0.        0.                    0.
 JAN         76.        0.        0.        0.        0.        0.                    0.
 FEB         32.        0.        0.        0.        0.        0.                    0.
 MAR        504.        2.        0.        0.        0.        0.                    0.
 APR       2128.      457.      211.       46.       21.       11.                  190.
 MAY        200.      200.      200.      200.      200.      200.                    0.
 JUNE      2718.      346.      123.       16.        6.        2.                  117.
 JULY      1923.       10.        1.        0.        0.        0.                    1.
 AUG         54.        0.        0.        0.        0.        0.                    0.
 SEPT         1.        0.        0.        0.        0.        0.                    0.
 ANNUAL     195.      132.      109.       74.       61.       52.                   48.

           Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  CP18
                   YAMPA RIVER BELOW PROPOSED JUNIPER RESERVOIR
                        SIMULATION OF MONTHLY FLOWS

          DATA SKEW K FACTORS

 MONTH      SKEW                                                RETURN PERIOD

                          10 YRS          6.67 YRS          2 YRS          1.25 YRS          1.11 YRS          1.05 YRS

OCT         2.42          1.2639            .1041          -.3494           -.7272            -.7988            -.8239
NOV         2.42          1.2643            .1046          -.3491           -.7277            -.7996            -.8249
DEC         1.92          1.3117            .1788          -.2914           -.7904            -.9255            -.9918
JAN         1.02          1.3398            .3085          -.1613           -.8523           -1.1312           -1.3231
FEB         1.26          1.3406            .2862          -.1860           -.8464           -1.0986           -1.2653
MAR          .51          1.3222            .3731          -.0806           -.8559           -1.2181           -1.4956
APR        -6.65           .6600            .5184           .3960           -.4200           -1.1800           -2.0030
MAY         1.03          1.3397            .3104          -.1590           -.8526           -1.1339           -1.3282
JUNE       -4.84           .6600            .5184           .3960           -.4200           -1.1800           -2.0030
JULY        -.17          1.2535            .4531           .0389           -.8279           -1.3038           -1.7090
AUG          .90          1.3390            .3197          -.1480           -.8540           -1.1470           -1.3530
SEPT        2.14          1.2972            .1514          -.3142           -.7698            -.8793            -.9279
ANNUAL       .10          1.2917            .4183          -.0166           -.8459           -1.2703           -1.6168

FREQUENCY OF FLOWS

                              YEARS FLOW IS NOT EXCEEDED

          MONTH   9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20            Q2 - Q10

         OCT           1.        0.        0.        0.        0.        0.                 0.
         NOV           1.        0.        0.        0.        0.        0.                 0.
         DEC           4.        0.        0.        0.        0.        0.                 0.
         JAN         108.        0.        0.        0.        0.        0.                 0.
         FEB          44.        0.        0.        0.        0.        0.                 0.
         MAR         652.        2.        0.        0.        0.        0.                 0.
         APR         693.      537.      430.       99.       25.        6.               405.
         MAY         200.      200.      200.      200.      200.      200.                 0.
         JUNE        605.      429.      320.       44.        7.        1.               312.
         JULY       1614.       12.        1.        0.        0.        0.                 1.
         AUG          75.        0.        0.        0.        0.        0.                 0.
         SEPT          1.        0.        0.        0.        0.        0.                 0.
         ANNUAL      195.      131.      108.       74.       61.       52.                47.

                    Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LITTLE SNAKE RIVER NEAR LILY, COLORADO
MEASURED FLOWS IN CUBIC FEET PER SECOND
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1927	142.	84.	81.	81.	72.	228.	1835.	3899.	1875.	597.	53.	77.	755.
1928	189.	363.	244.	179.	191.	785.	1468.	4230.	2067.	115.	13.	16.	825.
1929	303.	166.	120.	150.	150.	950.	2000.	5539.	4116.	581.	167.	260.	1212.
1930	255.	147.	85.	70.	120.	255.	1283.	1471.	973.	54.	94.	83.	407.
1931	101.	25.	25.	67.	145.	620.	2074.	2268.	1367.	455.	85.	15.	604.
1932	176.	260.	140.	205.	210.	1135.	2344.	4505.	2919.	529.	56.	43.	1046.
1933	136.	182.	90.	16.	18.	390.	1116.	3056.	3642.	254.	10.	3.	743.
1934	1.	34.	65.	16.	18.	200.	465.	477.	37.	0.	0.	0.	110.
1935	0.	0.	33.	65.	36.	179.	397.	1431.	1666.	114.	28.	55.	334.
1936	2.	59.	130.	98.	108.	311.	1233.	2824.	958.	65.	237.	3.	505.
1937	59.	105.	49.	49.	18.	211.	1175.	3394.	2052.	732.	82.	103.	672.
1938	71.	114.	111.	128.	155.	320.	1197.	3391.	2059.	184.	42.	171.	663.
1939	93.	183.	175.	109.	137.	544.	902.	2196.	646.	6.	0.	6.	418.
1940	43.	47.	36.	22.	46.	225.	841.	2722.	767.	8.	4.	29.	359.
1941	202.	70.	35.	40.	56.	338.	779.	2947.	1265.	87.	534.	145.	545.
1942	323.	242.	178.	118.	112.	447.	1801.	2247.	1986.	176.	4.	0.	636.
1943	48.	70.	47.	66.	93.	230.	1237.	1399.	2274.	144.	43.	7.	470.
1944	17.	66.	73.	65.	72.	127.	622.	2446.	2737.	242.	7.	0.	540.
1945	80.	59.	78.	66.	90.	160.	640.	2689.	2957.	823.	213.	73.	662.
1946	102.	136.	86.	69.	139.	325.	1504.	1826.	1058.	68.	50.	4.	447.
1947	154.	179.	91.	67.	95.	534.	966.	3150.	1919.	348.	84.	120.	645.
1948	177.	141.	72.	60.	60.	85.	640.	2304.	1052.	100.	2.	0.	393.
1949	22.	101.	115.	114.	118.	248.	1335.	3597.	2797.	396.	20.	9.	741.
1950	193.	126.	101.	78.	80.	285.	1172.	2588.	2374.	290.	11.	16.	610.
1951	53.	102.	73.	61.	145.	239.	597.	1970.	1452.	164.	15.	4.	407.
1952	243.	92.	81.	71.	83.	89.	3259.	4817.	2927.	241.	123.	31.	1005.
1953	31.	51.	65.	75.	75.	308.	431.	1441.	1828.	84.	60.	2.	371.
1954	9.	94.	72.	75.	118.	216.	885.	1112.	305.	25.	0.	41.	246.
1955	45.	63.	68.	57.	59.	272.	587.	1600.	961.	77.	58.	2.	322.
1956	3.	69.	146.	115.	89.	1240.	1172.	2611.	1200.	38.	82.	1.	567.
1957	7.	63.	54.	59.	80.	210.	591.	2438.	3632.	1100.	129.	45.	701.
1958	103.	150.	109.	94.	165.	309.	777.	3548.	1666.	69.	6.	17.	587.
1959	29.	62.	61.	58.	83.	253.	421.	1128.	1002.	146.	64.	270.	298.
1960	174.	162.	85.	51.	68.	151.	1474.	1534.	1222.	53.	3.	4.	414.
1961	34.	44.	44.	36.	54.	279.	320.	1016.	795.	5.	10.	57.	225.
1962	213.	159.	155.	139.	452.	1260.	2663.	2727.	1412.	249.	6.	0.	786.
1963	14.	53.	59.	46.	124.	232.	483.	1460.	747.	18.	76.	57.	281.
1964	7.	51.	40.	51.	57.	81.	390.	2338.	1918.	294.	21.	4.	439.
1965	6.	75.	102.	92.	95.	191.	957.	2591.	2728.	612.	178.	314.	662.
1966	224.	146.	130.	135.	96.	1119.	1006.	1542.	616.	28.	4.	0.	423.
1967	54.	67.	89.	63.	80.	341.	524.	1691.	2298.	448.	31.	59.	479.
1968	75.	65.	104.	101.	128.	262.	591.	2882.	3243.	367.	94.	29.	662.
1969	91.	105.	92.	91.	87.	429.	1482.	2664.	1271.	296.	30.	43.	558.

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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  09260000
                LITTLE SNAKE RIVER NEAR LILY, COLORADO
                     MEASURED  FLOWS IN CUBIC FEET PER SECOND

          DATA SKEW K FACTORS

 MONTH      SKEW                                               RETURN PERIOD

                          10 YRS          6.67 YRS            2 YRS          1.25 YRS          1.11 YRS          1.05 YRS

OCT        -3.26           .6600            .5184             .3960          -.4200           -1.1800           -2.0030
NOV        -5.97           .6600            .5184             .3960          -.4200           -1.1800           -2.0030
DEC         -.40          1.2307            .4686             .0664          -.8158           -1.3171           -1.7506
JAN         -.94          1.1154            .5184             .1736          -.7502           -1.3406           -1.8872
FEB         -.39          1.2298            .4691             .0673          -.8154           -1.3175           -1.7519
MAR          .39          1.3016            .4063            -.0342          -.8502           -1.2571           -1.5839
APR          .11          1.2907            .4195            -.0148          -.8455           -1.2716           -1.6198
MAY         -.85          1.1372            .5116             .1563          -.7633           -1.3395           -1.8678
JUNE       -2.55           .7345            .5333             .3723          -.4881           -1.2304           -2.0125
JULY       -1.66           .9602            .5443             .2735          -.6534           -1.3217           -1.9755
AUG        -2.36           .7863            .5407             .3543          -.5301           -1.2576           -2.0114
SEPT       -1.77           .9377            .5457             .2855          -.6383           -1.3157           -1.9833
ANNUAL     -1.04          1.0945            .5238             .1889          -.7373           -1.3404           -1.9035

FREQUENCY OF FLOWS

                              YEARS FLOW IS NOT EXCEEDED

          MONTH   9 IN 10   2 IN 3   1 IN 2   1 IN 5  1 IN 10  1 IN 20          Q2 - Q10

          OCT        184.     138.     107.      20.       4.       1.              103.
          NOV        245.     192.     156.      38.      10.       3.              145.
          DEC        153.     106.      88.      57.      45.      37.               43.
          JAN        136.      99.      83.      50.      37.      28.               46.
          FEB        216.     131.     100.      56.      40.      30.               60.
          MAR        725.     399.     297.     172.     131.     106.              166.
          APR       1891.    1170.     921.     582.     461.     380.              460.
          MAY       3955.    2964.    2515.    1646.    1261.     988.             1254.
          JUNE      2747.    2353.    2078.    1070.     604.     331.             1474.
          JULY       629.     327.     214.      50.      17.       6.              196.
          AUG        132.      75.      49.       6.       1.       0.               48.
          SEPT       159.      49.      22.       1.       0.       0.               22.
          ANNUAL     841.     658.     570.     383.     295.     232.              275.

                    Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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STATISTICAL PARAMETERS FOR STATION   09260000     LITTLE SNAKE RIVER NEAR LILY. COLORADO
  MEASURED  FLOWS IN CUBIC FEET PER SECOND

                                   ARTH.                     LOG PARAMETERS
                        MONTH     AVERAGE       MEAN    VARIANCE   STD. DEV.      SKEW

             1          OCT        101.46      1.6760      .7961       .8923    -3.2628
             2          NOV        112.75      1.8974      .5565       .7460    -5.9743
             3          DEC         94.32      1.9284      .0436       .2088     -.3977
             4          JAN         84.88      1.8763      .0535       .2313     -.9400
             5          FEB        117.64      1.9814      .0822       .2868     -.3922
             6          MAR        383.96      2.4829      .0840       .2898      .3931
             7          APR       1077.96      2.9677      .0573       .2394      .1131
             8          MAY       2569.34      3.3693      .0402       .2004     -.8482
             9          JUNE      1885.27      3.1930      .1121       .3348    -2.5457
            10          JULY       275.04      2.1431      .4656       .6823    -1.6609
            11          AUG         64.90      1.3364      .9985       .9992    -2.3635
            12          SEPT        50.24       .9710     1.7200      1.3115    -1.7708
            13          ANNUAL     569.20      2.7205      .0349       .1867    -1.0405

            SAMPLE SIZE   50 YEARS


LOG NORMAL DISTRIBUTION STATION   09260000     LITTLE SNAKE RIVER NEAR LILY. COLORADO
  MEASURED  FLOWS IN CUBIC FEET PER SECOND

                                YEARS FLOW IS NOT EXCEEDED

           MONTH   9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20          Q2 - Q10

          OCT         660.      114.       47.        8.        3.        2.               44.
          NOV         714.      165.       79.       19.        9.        5.               70.
          DEC         157.      104.       85.       57.       46.       38.               39.
          JAN         149.       95.       75.       48.       38.       31.               37.
          FEB         223.      127.       96.       55.       41.       32.               55.
          MAR         715.      405.      304.      173.      129.      101.              175.
          APR        1882.     1176.      928.      584.      458.      375.              470.
          MAY        4229.     2853.     2340.     1587.     1295.     1096.             1045.
          JUNE       4190.     2171.     1560.      815.      580.      439.              979.
          JULY       1042.      273.      139.       37.       19.       10.              120.
          AUG         414.       58.       22.        3.        1.        0.               21.
          SEPT        449.       34.        9.        1.        0.        0.                9.
          ANNUAL      912.      632.      525.      366.      303.      259.              223.

                 Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LITTLE SNAKE RIVER NEAR LILY. COLORADO (CONTINUED)

1970	148.	135.	86.	94.	190.	241.	651.	3503.	2979.	464.	43.	63.	718.
1971	183.	167.	133.	141.	159.	612.	1607.	3370.	3606.	584.	25.	55.	887.
1972	123.	147.	154.	140.	424.	752.	808.	1697.	1625.	102.	7.	3.	497.
1973	73.	153.	106.	100.	91.	127.	1061.	3572.	2517.	586.	109.	84.	717.
1974	98.	166.	132.	106.	113.	189.	1073.	4140.	2540.	281.	46.	7.	743.
1975	57.	104.	78.	71.	96.	246.	409.	2605.	2526.	758.	119.	76.	597.
1976	88.	134.	139.	124.	330.	419.	653.	2374.	1685.	296.	68.	9.	527.
AVERAGE	101.	113.	94.	85.	118.	384.	1078.	2569.	1885.	275.	65.	50.	569.
Q/QANN	1.513	1.627	1.406	1.266	1.599	5.725	15.555	38.311	27.204	4.101	.968	.725	100.000
COV VAR	.834	.585	.460	.468	.728	.782	.579	.412	.506	.926	1.359	1.422	.380
SKEW	.777	1.314	.970	.777	2.430	1.752	1.389	.537	.324	1.152	3.433	2.273	.546
MAXIMUM	323.	363.	244.	205.	452.	1260.	3259.	5539.	4116.	1100.	534.	314.	1212.
MINIMUM	0.	0.	25.	16.	18.	81.	320.	477.	37.	0.	0.	0.	110.

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YAMPA RIVER AT MAYBELL, COLORADO
MEASURED FLOWS IN CUBIC FEET PER SECOND
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1927	306.	305.	330.	320.	310.	590.	3189.	8261.	6118.	1564.	508.	356.	1851.
1928	499.	673.	540.	420.	520.	1445.	2879.	9963.	5733.	1401.	470.	362.	2083.
1929	409.	476.	380.	400.	400.	1900.	4941.	11270.	8953.	2566.	792.	972.	2795.
1930	697.	570.	480.	340.	380.	785.	4349.	4396.	4022.	569.	623.	437.	1469.
1931	682.	691.	505.	325.	360.	740.	2750.	3864.	2974.	468.	193.	152.	1142.
1932	300.	300.	247.	177.	195.	915.	3361.	8282.	6167.	2073.	631.	259.	1915.
1933	341.	392.	245.	170.	300.	470.	1890.	4975.	7573.	822.	269.	163.	1465.
1934	183.	191.	195.	115.	340.	535.	1567.	2450.	548.	20.	27.	28.	517.
1935	121.	195.	180.	262.	368.	455.	1173.	3886.	6336.	1174.	283.	139.	1213.
1936	169.	234.	215.	240.	270.	440.	4285.	7634.	4204.	730.	352.	136.	1579.
1937	210.	201.	180.	175.	260.	600.	1673.	6436.	4098.	1208.	288.	177.	1297.
1938	315.	353.	376.	357.	434.	818.	2854.	6889.	6100.	1121.	311.	401.	1696.
1939	247.	314.	355.	252.	254.	1216.	2964.	6136.	2926.	338.	144.	217.	1284.
1940	304.	250.	162.	154.	249.	563.	2412.	6076.	3264.	391.	74.	91.	1169.
1941	345.	312.	248.	221.	296.	639.	1546.	7338.	4036.	811.	339.	191.	1366.
1942	654.	474.	391.	343.	345.	815.	4009.	5874.	5524.	952.	226.	84.	1641.
1943	171.	247.	208.	188.	244.	753.	3190.	3850.	4539.	1143.	323.	144.	1249.
1944	166.	239.	199.	168.	179.	292.	735.	5064.	5827.	1039.	131.	33.	1174.
1945	142.	268.	208.	197.	177.	381.	1497.	7143.	6611.	2648.	915.	333.	1716.
1946	274.	319.	263.	227.	330.	645.	3614.	3584.	3831.	704.	261.	149.	1181.
1947	333.	413.	319.	225.	269.	1233.	2760.	8007.	5327.	1929.	527.	278.	1809.
1948	352.	445.	624.	610.	644.	924.	3277.	7368.	4035.	837.	319.	97.	1631.
1949	263.	278.	258.	250.	288.	711.	3223.	6868.	7270.	1953.	334.	178.	1824.
1950	391.	335.	252.	245.	277.	448.	2239.	4410.	5497.	1273.	218.	191.	1314.
1951	250.	287.	268.	235.	268.	533.	1852.	5356.	5299.	1761.	475.	214.	1403.
1952	334.	265.	202.	228.	252.	306.	4033.	8394.	7971.	1186.	542.	243.	1997.
1953	190.	208.	222.	237.	220.	401.	1218.	3602.	6116.	842.	396.	102.	1145.
1954	151.	264.	189.	237.	272.	382.	1728.	3398.	1436.	273.	122.	183.	721.
1955	378.	273.	204.	202.	191.	456.	2009.	4881.	3355.	503.	248.	67.	1067.
1956	133.	263.	322.	277.	246.	489.	3598.	6518.	4358.	539.	280.	64.	1426.
1957	126.	213.	187.	206.	234.	467.	2108.	7156.	11430.	5819.	1052.	450.	2459.
1958	469.	483.	405.	328.	504.	675.	2716.	8931.	5539.	573.	178.	165.	1751.
1959	216.	246.	215.	220.	262.	350.	1501.	4306.	4827.	783.	351.	205.	1124.
1960	665.	532.	377.	235.	221.	663.	4035.	4675.	4496.	592.	155.	97.	1394.
1961	197.	254.	211.	199.	215.	309.	948.	3790.	3272.	356.	131.	534.	869.
1962	1001.	566.	353.	322.	743.	733.	6496.	7145.	5119.	1841.	295.	113.	2059.
1963	288.	288.	217.	213.	391.	466.	1324.	4081.	2475.	270.	215.	199.	870.
1964	117.	201.	137.	137.	160.	221.	1128.	5428.	4916.	1348.	317.	177.	1194.
1965	172.	231.	273.	270.	266.	285.	2626.	6280.	7648.	2439.	753.	501.	1813.
1966	604.	421.	326.	335.	280.	1427.	2044.	3858.	1848.	285.	123.	48.	970.
1967	268.	218.	206.	192.	228.	691.	1486.	4063.	5305.	1766.	347.	237.	1252.
1968	273.	246.	195.	222.	244.	454.	1494.	5584.	7832.	1532.	598.	256.	1578.
1969	354.	325.	278.	283.	284.	419.	4173.	6510.	3732.	1216.	340.	337.	1523.

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YAMPA RIVER AT MAYBELL, COLORADO (CONTINUED)

1970	450.	412.	360.	350.	394.	489.	1374.	8302.	7449.	1992.	442.	287.	1864.
1971	470.	437.	345.	374.	384.	1081.	4649.	6401.	7756.	1901.	320.	254.	2030.
1972	306.	355.	310.	347.	436.	1175.	2116.	4248.	4872.	538.	151.	197.	1253.
1973	400.	382.	351.	305.	284.	428.	1626.	7689.	6022.	2128.	517.	205.	1701.
1974	250.	338.	357.	308.	271.	577.	3775.	9695.	6208.	1236.	314.	89.	1957.
1975	207.	280.	138.	220.	298.	458.	1566.	5439.	7270.	3388.	509.	180.	1666.
1976	247.	298.	272.	246.	343.	531.	1463.	5011.	3712.	997.	357.	165.	1140.
AVERAGE	328.	335.	286.	262.	312.	656.	2589.	6015.	5236.	1277.	362.	223.	1492.
Q/QANN	1.864	1.845	1.624	1.491	1.615	3.729	14.252	34.214	28.818	7.262	2.057	1.226	100.000
COV VAR	.549	.363	.367	.328	.360	.520	.478	.324	.385	.767	.588	.720	.290
SKEW	1.568	1.235	1.129	1.436	1.837	1.636	.821	.497	.322	2.291	1.203	2.384	.456
MAXIMUM	1001.	691.	624.	610.	743.	1900.	6496.	11270.	11430.	5819.	1052.	972.	2795.
MINIMUM	117.	191.	137.	115.	160.	221.	735.	2450.	548.	20.	27.	28.	517.

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STATISTICAL PARAMETERS FOR STATION   09251000     YAMPA RIVER AT MAYBELL, COLORADO
  MEASURED FLOWS IN CUBIC FEET PER SECOND


                                  ARITH.                  LOG PARAMETERS
                  MONTH          AVERAGE        MEAN   VARIANCE  STD. DEV.     SKEW


          1        OCT            327.80      2.4602     .0477     .2184      .2398
          2        NOV            335.22      2.5008     .0206     .1434      .5849
          3        DEC            285.60      2.4296     .0225     .1502      .2929
          4        JAN            262.18      2.3479     .0180     .1343      .1505
          5        FEB            311.60      2.4710     .0186     .1365      .7031
          6        MAR            655.58      2.7690     .0399     .1997      .4250
          7        APR           2589.26      3.3642     .0447     .2115     -.1107
          8        MAY           6015.30      3.7565     .0207     .1439     -.1812
          9        JUNE          5235.52      3.6774     .0472     .2173    -1.9386
         10        JULY          1276.77      2.9807     .1472     .3837    -1.6247
         11        AUG            361.72      2.4789     .0831     .2883    -1.0081
         12        SEPT           222.73      2.2530     .0903     .3004     -.4325
         13        ANNUAL        1492.17      3.1548     .0180     .1340     -.6993


          SAMPLE SIZE   50 YEARS



LOG NORMAL DISTRIBUTION STATION   09251000     YAMPA RIVER AT MAYBELL, COLORADO
  MEASURED FLOWS IN CUBIC FEET PER SECOND


                             YEARS FLOW IS NOT EXCEEDED

     MONTH    9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20          Q2 - Q10

    OCT          550.      358.      289.      189.      151.      126.              137.
    NOV          484.      365.      317.      240.      208.      184.              109.
    DEC          419.      312.      269.      201.      173.      152.               96.
    JAN          372.      285.      250.      193.      168.      150.               82.
    FEB          443.      338.      296.      227.      198.      176.               98.
    MAR         1059.      716.      587.      399.      326.      276.              262.
    APR         4319.     2851.     2313.     1535.     1239.     1038.             1074.
    MAY         8729.     6579.     5708.     4318.     3732.     3309.             1975.
    JUNE        9037.     5897.     4758.     3122.     2505.     2089.             2253.
    JULY        2969.     1397.      957.      455.      308.      224.              648.
    AUG          705.      400.      301.      172.      129.      101.              173.
    SEPT         435.      241.      179.      100.       74.       57.              105.
    ANNUAL      2121.     1630.     1428.     1101.      962.      860.              467.

                   Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  09251000
                     YAMPA RIVER AT MAYBELL, COLORADO
                          MEASURED FLOWS IN CUBIC FEET PER SECOND

          DATA SKEW K FACTORS

 MONTH      SKEW                                           RETURN PERIOD

                          10 YRS          6.67 YRS           2 YRS          1.25 YRS          1.11 YRS         1.05 YRS

OCT          .24          1.2974            .4115           -.0266           -.8484           -1.2628          -1.5979
NOV          .58          1.3179            .3820           -.0686           -.8552           -1.2287          -1.5190
DEC          .29          1.2926            .4173           -.0181           -.8463           -1.2692          -1.6139
JAN          .15          1.2869            .4238           -.0084           -.8440           -1.2761          -1.6306
FEB          .70          1.3328            .3465           -.1155           -.8570           -1.1835          -1.4241
MAR          .43          1.3150            .3868           -.0620           -.8545           -1.2345          -1.5318
APR         -.11          1.2464            .4585            .0482           -.8246           -1.3081          -1.7232
MAY         -.18          1.2556            .4515            .0362           -.8289           -1.3025          -1.7049
JUNE       -1.94           .8790            .5466            .3144           -.5986           -1.2971          -1.9991
JULY       -1.62           .9512            .5451            .2785           -.6472           -1.3195          -1.9788
AUG        -1.01          1.0877            .5256            .1938           -.7331           -1.3401          -1.9087
SEPT        -.43          1.2052            .4829            .0938           -.8026           -1.3264          -1.7895
ANNUAL      -.70          1.1829            .4940            .1161           -.7899           -1.3330          -1.8191

FREQUENCY OF FLOWS

                                     YEARS FLOW IS NOT EXCEEDED

          MONTH    9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20              Q2 - Q10

          OCT         554.      355.      285.      188.      153.      129.                  132.
          NOV         490.      359.      310.      239.      211.      192.                   99.
          DEC         420.      311.      267.      201.      173.      154.                   94.
          JAN         372.      285.      249.      193.      168.      151.                   81.
          FEB         450.      330.      285.      226.      204.      189.                   81.
          MAR        1076.      702.      571.      397.      333.      290.                  238.
          APR        4245.     2892.     2368.     1548.     1223.      999.                 1145.
          MAY        8652.     6629.     5776.     4337.     3707.     3244.                 2070.
          JUNE       7386.     6254.     5568.     3526.     2486.     1750.                 3082.
          JULY       2217.     1548.     1224.      540.      298.      167.                  925.
          AUG         620.      427.      343.      185.      124.       85.                  219.
          SEPT        412.      250.      191.      103.       72.       52.                  120.
          ANNUAL     2057.     1663.     1480.     1119.      947.      815.                  534.

                 Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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YAMPA RIVER BELOW PROPOSED CROSS MOUNTAIN DAM
SIMULATED MONTHLY FLOWS
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1927	57.	52.	57.	59.	57.	517.	344.	1731.	3421.	3226.	2342.	507.	1038.
1928	517.	380.	501.	568.	556.	344.	344.	2479.	3357.	3191.	2336.	1464.	1342.
1929	401.	581.	627.	597.	597.	344.	344.	2859.	3923.	3408.	2394.	1587.	1477.
1930	600.	600.	663.	678.	627.	490.	494.	2158.	3043.	3054.	2369.	1022.	1322.
1931	607.	724.	725.	681.	602.	344.	344.	1720.	2005.	1720.	382.	246.	844.
1932	0.	52.	42.	33.	36.	166.	344.	1720.	3256.	3318.	2365.	960.	1030.
1933	571.	565.	658.	375.	55.	353.	344.	1720.	2855.	3087.	457.	577.	974.
1934	698.	550.	34.	22.	62.	97.	362.	1720.	1343.	0.	0.	0.	412.
1935	0.	32.	30.	48.	67.	83.	431.	1720.	2305.	2427.	440.	528.	679.
1936	442.	39.	37.	44.	51.	80.	344.	1720.	2941.	3083.	1205.	574.	885.
1937	650.	605.	31.	32.	48.	109.	344.	1720.	2424.	2552.	385.	479.	745.
1938	577.	62.	67.	64.	79.	423.	344.	1720.	3186.	3134.	1069.	409.	933.
1939	615.	564.	373.	46.	47.	344.	344.	1720.	2570.	2097.	462.	565.	816.
1940	394.	42.	27.	29.	46.	102.	344.	1720.	2240.	1885.	464.	554.	657.
1941	377.	54.	43.	41.	54.	116.	344.	1720.	2577.	2776.	344.	436.	744.
1942	381.	448.	568.	211.	63.	344.	344.	1720.	3097.	3110.	592.	566.	959.
1943	661.	598.	35.	34.	45.	137.	344.	1720.	2119.	2293.	424.	574.	752.
1944	414.	40.	34.	31.	33.	54.	344.	1720.	2426.	2493.	458.	569.	722.
1945	323.	46.	35.	36.	34.	71.	344.	1720.	3000.	3422.	1766.	510.	949.
1946	606.	611.	209.	41.	59.	418.	344.	1720.	2256.	1720.	417.	569.	751.
1947	453.	72.	55.	41.	49.	344.	344.	1759.	3292.	3292.	2051.	462.	1025.
1948	529.	606.	439.	689.	689.	663.	344.	1828.	3044.	3090.	456.	576.	1082.
1949	675.	284.	45.	46.	53.	129.	344.	1720.	3177.	3290.	2304.	564.	1060.
1950	513.	621.	243.	670.	165.	82.	344.	1720.	2323.	2480.	448.	551.	851.
1951	419.	50.	47.	43.	49.	506.	344.	1720.	2504.	3256.	447.	571.	835.
1952	462.	58.	36.	42.	665.	176.	344.	1720.	3729.	3153.	2349.	1443.	1183.
1953	673.	698.	684.	47.	40.	73.	393.	1720.	2231.	2180.	407.	575.	814.
1954	447.	46.	32.	44.	49.	70.	344.	1720.	1720.	812.	0.	0.	442.
1955	5.	47.	35.	692.	138.	82.	344.	1720.	2021.	1720.	409.	147.	617.
1956	0.	45.	57.	51.	45.	88.	344.	1720.	2641.	3036.	384.	578.	753.
1957	687.	377.	33.	38.	43.	85.	344.	1720.	3912.	4027.	2440.	1480.	1272.
1958	605.	597.	639.	654.	582.	434.	344.	2254.	3309.	3035.	2276.	1435.	1352.
1959	681.	686.	688.	691.	293.	64.	397.	1720.	2120.	1720.	403.	344.	820.
1960	57.	585.	238.	698.	160.	120.	344.	1720.	2407.	2585.	465.	579.	833.
1961	330.	44.	36.	36.	40.	57.	492.	1720.	1720.	1505.	0.	0.	501.
1962	493.	588.	592.	209.	344.	344.	344.	2362.	3234.	3263.	2297.	1191.	1278.
1963	688.	696.	690.	60.	70.	513.	344.	1720.	1721.	1517.	0.	0.	673.
1964	0.	34.	23.	26.	29.	41.	393.	1720.	2328.	2179.	447.	572.	652.
1965	412.	40.	48.	50.	49.	555.	344.	1720.	2624.	3384.	2393.	344.	1006.
1966	482.	601.	618.	612.	652.	344.	344.	1720.	1834.	1720.	213.	0.	763.
1967	0.	36.	36.	36.	42.	125.	344.	1720.	2206.	2644.	436.	524.	683.
1968	500.	42.	34.	41.	620.	200.	344.	1720.	2890.	3214.	625.	553.	900.
1969	617.	642.	135.	52.	52.	75.	344.	1720.	2641.	3165.	488.	540.	877.

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YAMPA RIVER BELOW PROPOSED CROSS MOUNTAIN DAM (CONTINUED)

1970	559.	612.	662.	350.	72.	504.	344.	1720.	3405.	3297.	2337.	1457.	1284.
1971	524.	580.	614.	606.	588.	344.	344.	2078.	3709.	3274.	2302.	1451.	1372.
1972	432.	66.	62.	55.	657.	190.	344.	1720.	2982.	3326.	2344.	498.	1060.
1973	611.	581.	254.	641.	190.	105.	344.	1872.	3444.	3162.	2301.	876.	1205.
1974	652.	644.	671.	147.	55.	83.	377.	1720.	2784.	3549.	1991.	506.	1106.
1975	620.	614.	164.	45.	63.	97.	344.	1720.	2092.	2345.	398.	572.	760.
AVERAGE	449.	352.	259.	226.	199.	233.	356.	1805.	2703.	2678.	1104.	634.	921.
Q/QANN	4.141	3.137	2.390	2.084	1.673	2.150	3.175	16.637	24.105	24.679	10.172	5.658	100.000
COV VAR	.487	.786	1.047	1.190	1.195	.743	.094	.128	.229	.295	.842	.662	.275
SKEW	-1.086	-.121	.657	.940	1.202	.701	3.271	3.117	.052	-1.191	.470	.856	.284
MAXIMUM	698.	724.	725.	698.	689.	663.	494.	2859.	3923.	4027.	2440.	1587.	1477.
MINIMUM	0.	32.	23.	22.	29.	41.	344.	1720.	1393.	0.	0.	0.	412.

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STATISTICAL PARAMETERS FOR STATION   CP19        YAMPA RIVER BELOW PROPOSED CROSS MOUNTAIN DAM
  SIMULATED MONTHLY FLOWS


                                    ARTH.                   LOG PARAMETERS
                         MONTH     AVERAGE       MEAN   VARIANCE  STD. DEV.    SKEW


                1         OCT         449.33     2.0540     3.0867     1.7569   -2.5446
                2         NOV         351.78     2.2895      .3038      .5512    -.2931
                3         DEC         259.33     2.0920      .3194      .5651     .2546
                4         JAN         226.16     2.0115      .3000      .5478     .6300
                5         FEB         199.20     2.0179      .2258      .4752     .8152
                6         MAR         233.24     2.2389      .1200      .3464     .0940
                7         APR         355.98     2.5498      .0013      .0361    3.0898
                8         MAY        1805.31     3.2537      .0023      .0477    2.8707
                9         JUNE       2702.82     3.4200      .0109      .1042    -.4342
               10         JULY       2677.92     3.2883      .8585      .9266   -6.7818
               11         AUG        1103.71     2.4546     2.8190     1.6790   -2.9153
               12         SEPT        634.39     2.2039     3.1835     1.7842   -2.6458
               13         ANNUAL      920.96     2.9473      .0156      .1249    -.4480


              SAMPLE SIZE   49 YEARS



LOG NORMAL DISTRIBUTION STATION   CP19        YAMPA RIVER BELOW PROPOSED CROSS MOUNTAIN DAM
  SIMULATED MONTHLY FLOWS


                                YEARS FLOW IS NOT EXCEEDED

             MONTH    9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20            Q2 - Q10

            OCT        20246.      642.      113.        4.        1.        0.
            NOV          991.      336.      195.       67.       38.       24.               113.
            DEC          655.      216.      124.       41.       23.       15.               156.
            JAN          517.      176.      103.       36.       20.       13.               100.
            FEB          424.      167.      104.       41.       26.       17.                82.
            MAR          482.      244.      173.       89.       62.       47.                79.
            APR          395.      368.      355.      331.      319.      309.               111.
            MAY         2065.     1880.     1794.     1635.     1558.     1497.                36.
            JUNE        3578.     2915.     2630.     2149.     1933.     1772.               235.
            JULY       29931.     4850.     1942.      322.      126.       58.               697.
            AUG        40463.     1496.      285.       11.        2.        0.              1816.
            SEPT       30996.      932.      160.        5.        1.        0.               283.
            ANNUAL      1281.     1002.      886.      695.      613.      552.               159.
                                                                                            273.

                  Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  CP19
              YAMPA RIVER BELOW PROPOSED CROSS MOUNTAIN DAM
                   SIMULATED MONTHLY FLOWS

          DATA SKEW K FACTORS

 MONTH       SKEW                                                      RETURN PERIOD

                                10 YRS          6.67 YRS               2 YRS           1.25 YRS            1.11 YRS            1.05 YRS

OCT         -2.54               .7343           .5332                  .3724           -.4879              -1.2302             -2.0124
NOV          -.29              1.2440           .4601                  .0511           -.8235              -1.3095             -1.7276
DEC           .25              1.2961           .4131                 -.0243           -.8478              -1.2646             -1.6024
JAN           .63              1.3265           .3631                 -.0942           -.8567              -1.2048             -1.4679
FEB           .82              1.3355           .3351                 -.1296           -.8562              -1.1686             -1.3933
MAR           .09              1.2707           .4392                  .0160           -.8364              -1.2914             -1.6713
APR          3.09              1.1800           .0202                 -.3960           -.6360               -.6600              -.6650
MAY          2.87              1.2199           .0561                 -.3783           -.6766               -.7176              -.7287
JUNE         -.43              1.2055           .4827                  .0935           -.8027              -1.3263             -1.7891
JULY        -5.78               .6600           .5184                  .3960           -.4200              -1.1800             -2.0030
AUG         -2.92              2.0312           .7635                  .4163           -.3997               -.7190              -.8649
SEPT        -2.65               .7121           .5295                  .3803           -.4687              -1.2164             -2.0109
ANNUAL       -.45              1.2077           .4816                  .0913           -.8038              -1.3256             -1.7860

FREQUENCY OF FLOWS

                              YEARS FLOW IS NOT EXCEEDED

          MONTH   9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20              Q2 - Q10

          OCT       2208.      979.      511.       16.        1.        0.                  510.
          NOV        944.      349.      208.       68.       37.       22.                  171.
          DEC        668.      212.      120.       41.       24.       15.                   96.
          JAN        547.      162.       91.       35.       22.       16.                   69.
          FEB        449.      150.       90.       41.       29.       23.                   61.
          MAR        478.      246.      176.       89.       62.       46.                  114.
          APR        391.      355.      343.      336.      336.      336.                    7.
          MAY       2051.     1805.     1721.     1665.     1658.     1656.                   63.
          JUNE      3513.     2953.     2690.     2169.     1913.     1712.                  777.
          JULY      7940.     5869.     4521.      793.      157.       27.                 4364.
          AUG     732713.     5451.     1424.       61.       18.       10.                 1407.
          SEPT      2992.     1408.      763.       23.        1.        0.                  762.
          ANNUAL    1254.     1017.      909.      703.      605.      530.                  304.

                  Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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PGM J407 VER 3.4
(REV 10/22/79)                                      U. S. GEOLOGICAL SURVEY
                                               ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS
                                                FOLLOWING WRC GUIDELINES BULL. 17-A.


EXECUTION BEGINNING AT DATE, TIME =    5/13/80    134

INPUT FORMAT =   1  WATSTORE PEAK FILE RETRIEVAL


 EXPLANATION OF PEAK DISCHARGE QUALIFICATION CODES

 J407   FILE   MEANING

   D       3    DAM FAILURE, NON-RECURRENT FLOW ANOMALY
   G       8    DISCHARGE GREATER THAN STATED VALUE
   X      3+8   BOTH OF THE ABOVE
   L       4    DISCHARGE LESS THAN STATED VALUE
   K    6 OR C  KNOWN EFFECT OF REGULATION OR URBANIZATION
   H       7    HISTORIC PEAK


REPORT TROUBLE TO WATSTORE USER ASSISTANCE.
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PSM J407 VER 3.4                          U. S. GEOLOGICAL SURVEY
(REV 10/22/79)                     ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS
                                   FOLLOWING WRC GUIDELINES BULL. 17-A.            RUN-DATE  5/13/80 AT  134   SEQ  1.0001

 OPTIONS IN EFFECT --   PLOT  NORC  LGPT  NODR  PPOS  NORS  EXPR  CLIM

  STATION -        09251000/USGS    YAMPA RIVER NEAR MAYBELL, CO.                  1904-1978                        09251000/USGS

                                         I N P U T   D A T A   S U M M A R Y

   -- YEARS OF RECORD --     HISTORIC    GENERALIZED    SKEW       GAGE BASE     USER-SET OUTLIER CRITERIA
   SYSTEMATIC   HISTORIC      PEAKS         SKEW       OPTION      DISCHARGE     HIGH OUTLIER   LOW OUTLIER

       65          0            0          -0.300    WRC WEIGHTED     0.0            --             --

                              *********    NOTICE  --  PRELIMINARY MACHINE COMPUTATIONS.        *********
                              *********   USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION.   *********

   WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                     0.0
   WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.             2895.6
   WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS WERE NOTED.

                             ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

                                          FLOOD BASE                        LOGARITHMIC
                              FLOOD BASE  EXCEEDANCE     LOGARITHMIC        STANDARD       LOGARITHMIC
                              DISCHARGE   PROBABILITY       MEAN            DEVIATION         SKEW

           SYSTEMATIC RECORD     0.0        1.0000         3.9871            0.1350          -0.676
           W R C  ESTIMATE       0.0        1.0000         3.9871            0.1350          -0.501

                      ANNUAL FREQUENCY CURVE ORDINATES -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

                  ANNUAL                                          'EXPECTED-        95-PCT CONFIDENCE LIMITS
                EXCEEDANCE        W R C          SYSTEMATIC       PROBABILITY'       FOR  W R C  ESTIMATES
                PROBABILITY      ESTIMATE          RECORD          ESTIMATE          LOWER           UPPER

                  0.9950          3772.2           3589.6           3556.9           3167.3          4312.9
                  0.9900          4212.8           4057.1           4038.8           3598.4          4757.3
                  0.9500          5592.6           5524.9           5505.0           4980.4          6132.1
                  0.9000          6434.5           6416.9           6361.7           5837.2          6968.1
                  0.8000          7551.0           7588.4           7510.8           6973.3          8088.3
                  0.5000          9961.1          10050.5           9961.1           9346.7         10627.6
                  0.2000         12667.4          12469.8          12714.7          11816.2         13737.1
                  0.1000         14165.0          14040.0          14263.5          13114.6         15551.3
                  0.0400         15797.9          15462.5          15966.8          14494.1         17585.2
                  0.0200         16861.6          16346.1          17114.4          15376.8         18936.0
                  0.0100         17817.6          17110.1          18144.9          16161.5         20165.6
                  0.0050         18687.7          17779.4          19122.5          16869.3         21296.4
                  0.0020         19730.6          18547.9          20226.0          17710.5         22665.2
```

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PSM J407 VER 3.4
(REV 10/22/79)                         U. S. GEOLOGICAL SURVEY
                                  ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS
                                  FOLLOWING WRC GUIDELINES BULL. 17-A.           RUN-DATE  5/13/80 AT  134   SEQ  1.0001

 STATION -        09251000/USGS    YAMPA RIVER NEAR MAYBELL, CO.                  1904-1978                    09251000/USGS

                         *********    NOTICE  --  PRELIMINARY MACHINE COMPUTATIONS.        *********
                         *********   USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION.   *********

             INPUT DATA LISTING                        EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS
         WATER
          YEAR        DISCHARGE   CODES                WATER        RANKED        SYSTEMATIC         W R C
                                                        YEAR       DISCHARGE        RECORD         ESTIMATE
          1904          8050.0
          1905         11400.0                          1917        17900.0        0.0152          0.0152
          1916         11700.0                          1921        17700.0        0.0303          0.0303
          1917         17900.0                          1920        16000.0        0.0455          0.0455
          1918         10500.0                          1957        15700.0        0.0606          0.0606
          1919          7670.0                          1974        15400.0        0.0758          0.0758
          1920         16000.0                          1929        14400.0        0.0909          0.0909
          1921         17700.0                          1952        13800.0        0.1061          0.1061
          1922         10800.0                          1928        13700.0        0.1212          0.1212
          1923         10900.0                          1970        12700.0        0.1364          0.1364
          1924          7810.0                          1947        12400.0        0.1515          0.1515
          1925          6640.0                          1958        12200.0        0.1667          0.1667
          1926          9090.0                          1932        12100.0        0.1818          0.1818
          1927         11800.0                          1938        12100.0        0.1970          0.1970
          1928         13700.0                          1973        12100.0        0.2121          0.2121
          1929         14400.0                          1927        11800.0        0.2273          0.2273
          1930          7980.0                          1965        11800.0        0.2424          0.2424
          1931          6500.0                          1916        11700.0        0.2576          0.2576
          1932         12100.0                          1941        11700.0        0.2727          0.2727
          1933         11200.0                          1975        11700.0        0.2879          0.2879
          1934          4080.0                          1978        11600.0        0.3030          0.3030
          1935          9870.0                          1962        11500.0        0.3182          0.3182
          1936         10600.0                          1905        11400.0        0.3333          0.3333
          1937         10000.0                          1968        11400.0        0.3485          0.3485
          1938         12100.0                          1948        11300.0        0.3636          0.3636
          1939          7860.0                          1933        11200.0        0.3788          0.3788
          1940          9170.0                          1923        10900.0        0.3939          0.3939
          1941         11700.0                          1945        10900.0        0.4091          0.4091
          1942          9930.0                          1922        10800.0        0.4242          0.4242
          1943          9280.0                          1936        10600.0        0.4394          0.4394
          1944          9080.0                          1918        10500.0        0.4545          0.4545
          1945         10900.0                          1971        10300.0        0.4697          0.4697
          1946          6850.0                          1953        10100.0        0.4848          0.4848
          1947         12400.0                          1937        10000.0        0.5000          0.5000
          1948         11300.0                          1964         9990.0        0.5152          0.5152
          1949          9730.0                          1942         9930.0        0.5303          0.5303
          1950          8210.0                          1935         9870.0        0.5455          0.5455
          1951          8870.0                          1956         9870.0        0.5606          0.5606
          1952         13800.0                          1949         9730.0        0.5758          0.5758
          1953         10100.0                          1943         9280.0        0.5909          0.5909
                                                        1940         9170.0        0.6061          0.6061

                                    - CONTINUED --
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P3M J407 VER 3.4                              U. S. GEOLOGICAL SURVEY
(REV 10/22/79)                         ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS
                                        FOLLOWING WRC GUIDELINES BULL. 17-A.          RUN-DATE  5/13/80 AT  134   SEQ  1.0001

 STATION -        09251000/USGS    YAMPA RIVER NEAR MAYBELL, CO.                    1904-1978                         09251000/USGS

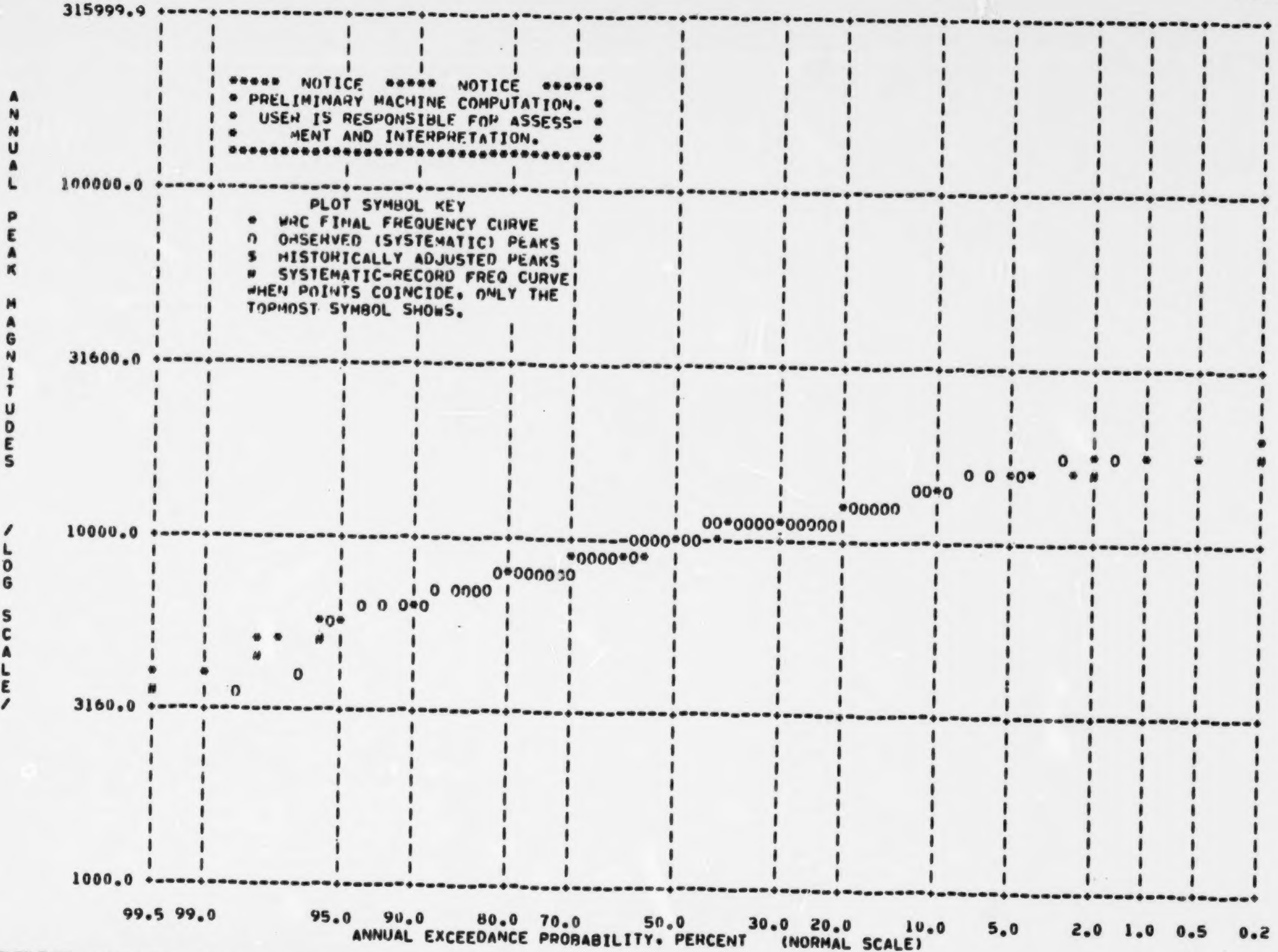
                                  *********    NOTICE  --  PRELIMINARY MACHINE COMPUTATIONS.       *********
                                  *********   USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION.   *********

                INPUT DATA LISTING                       EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

           WATER                                          WATER         RANKED          SYSTEMATIC          W R C
            YEAR         DISCHARGE   CODES                YEAR        DISCHARGE           RECORD          ESTIMATE

                                         -- CONTINUED --
            1954           5480.0                         1926          9090.0            0.6212            0.6212
            1955           7000.0                         1944          9080.0            0.6364            0.6364
            1956           9670.0                         1967          8890.0            0.6515            0.6515
            1957          15700.0                         1972          8890.0            0.6667            0.6667
            1958          12200.0                         1951          8870.0            0.6818            0.6818
            1959           6690.0                         1969          8290.0            0.6970            0.6970
            1960           8000.0                         1950          8210.0            0.7121            0.7121
            1961           6350.0                         1904          8050.0            0.7273            0.7273
            1962          11500.0                         1960          8000.0            0.7424            0.7424
            1963           6290.0                         1930          7980.0            0.7576            0.7576
            1964           9990.0                         1939          7860.0            0.7727            0.7727
            1965          11800.0                         1924          7810.0            0.7879            0.7879
            1966           6900.0                         1919          7670.0            0.8030            0.8030
            1967           8890.0                         1976          7450.0            0.8182            0.8182
            1968          11400.0                         1955          7000.0            0.8333            0.8333
            1969           8290.0                         1966          6900.0            0.8485            0.8485
            1970          12700.0                         1946          6850.0            0.8636            0.8636
            1971          10300.0                         1959          6690.0            0.8788            0.8788
            1972           8890.0                         1925          6640.0            0.8939            0.8939
            1973          12100.0                         1931          6500.0            0.9091            0.9091
            1974          15400.0                         1961          6350.0            0.9242            0.9242
            1975          11700.0                         1963          6290.0            0.9394            0.9394
            1976           7450.0                         1954          5480.0            0.9545            0.9545
            1977           3620.0                         1934          4080.0            0.9697            0.9697
            1978          11600.0                         1977          3620.0            0.9848            0.9848
```

```
PSM J407 VER 3.4                    U. S. GEOLOGICAL SURVEY
(REV 10/22/79)                ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS
                              FOLLOWING WRC GUIDELINES BULL. 17-A.          RUN-DATE  5/13/80 AT  134   SEQ  1.0001
  STATION -        09251000/USGS    YAMPA RIVER NEAR MAYBELL, CO.                  1904-1978                    09251000/USGS
```



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***** NOTICE ***** NOTICE ******
* PRELIMINARY MACHINE COMPUTATION. *
*  USER IS RESPONSIBLE FOR ASSESS- *
*     MENT AND INTERPRETATION.     *
************************************
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           PLOT SYMBOL KEY
*  WRC FINAL FREQUENCY CURVE
O  OBSERVED (SYSTEMATIC) PEAKS
$  HISTORICALLY ADJUSTED PEAKS
#  SYSTEMATIC-RECORD FREQ CURVE
WHEN POINTS COINCIDE, ONLY THE
TOPMOST SYMBOL SHOWS.
```

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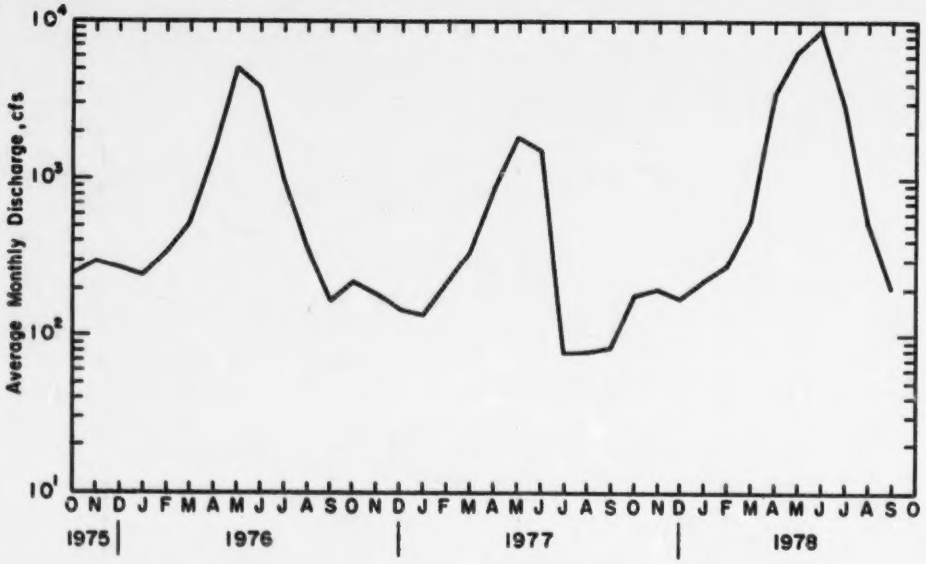
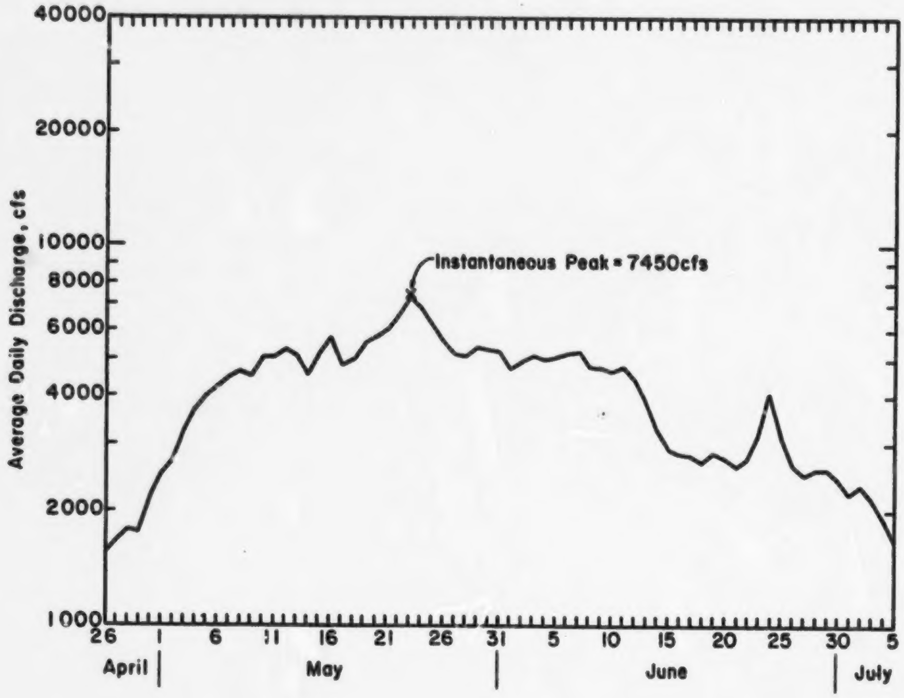
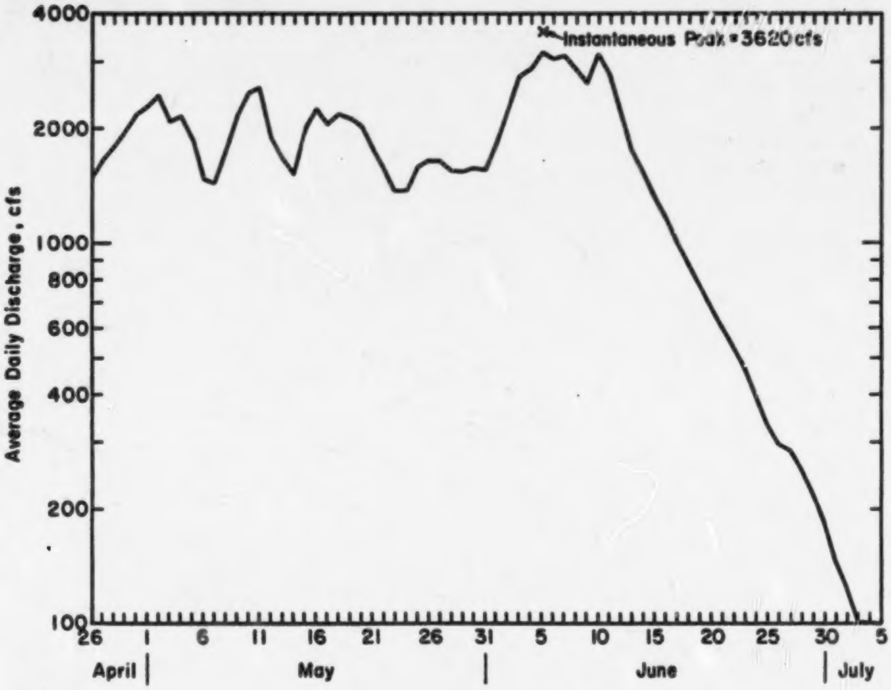


Average monthly discharge for Yampa River near Maybell, Colorado (October 1975-September 1978).

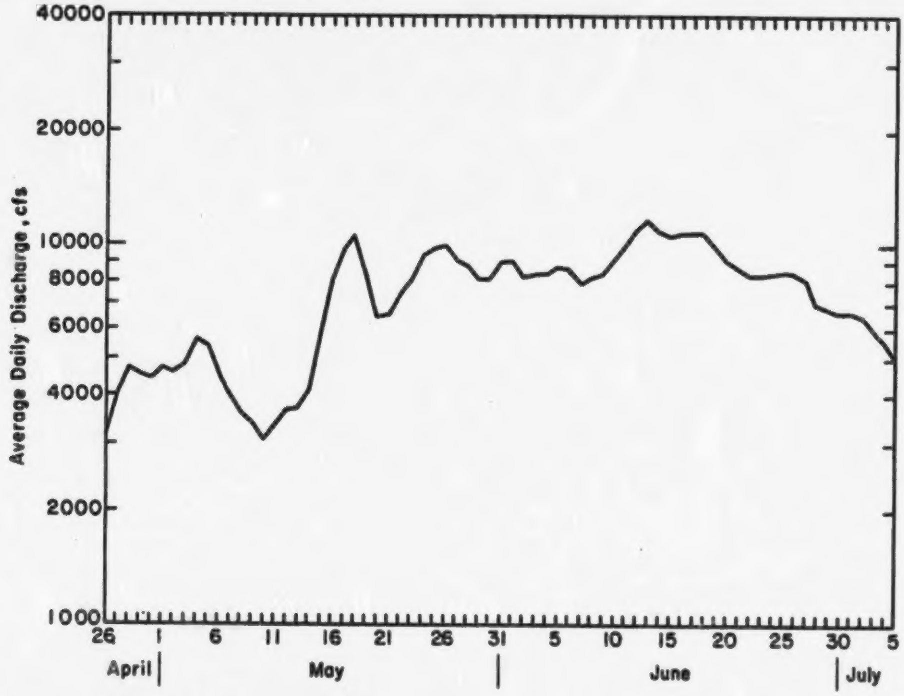
ANNUAL PEAK FLOW FREQUENCY ANALYSIS



Average daily discharges for Yampa River near Maybell, Colorado (April 26-July 5, 1976).



Average daily discharges for Yampa River near Maybell, Colorado (April 26-July 5, 1977).



Average daily discharges for Yampa River near Maybell, Colorado (April 26-July 5, 1978).

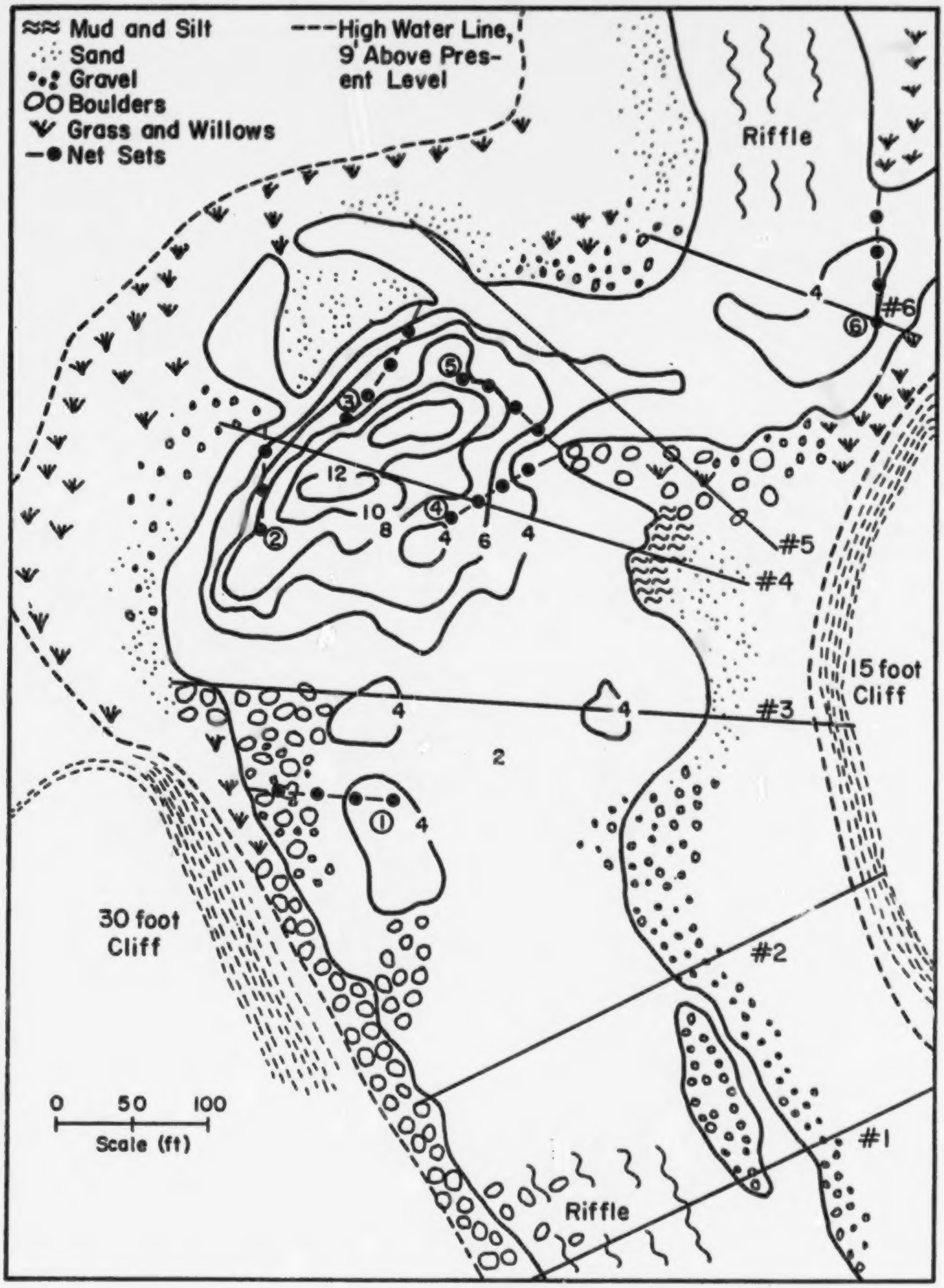
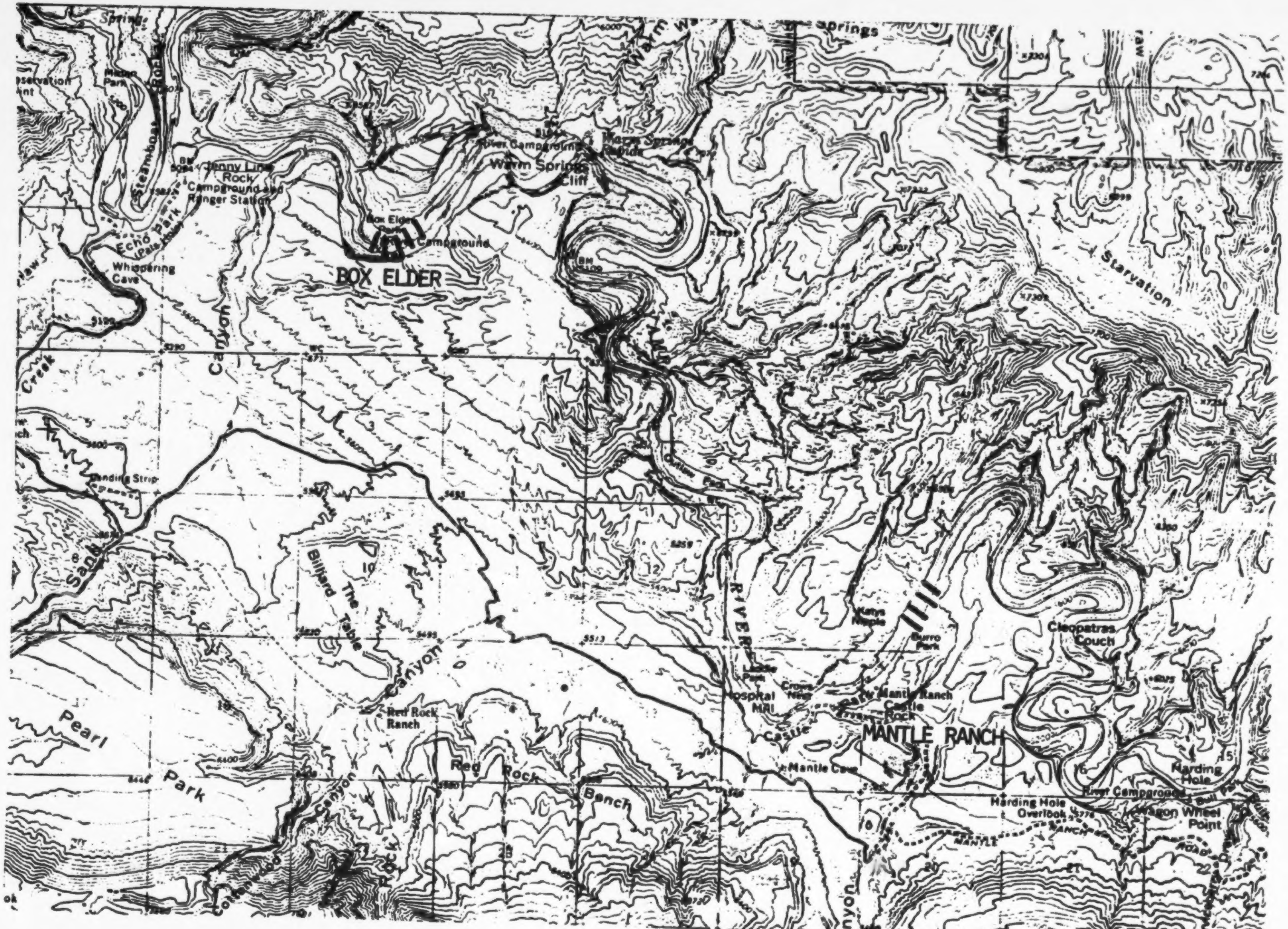
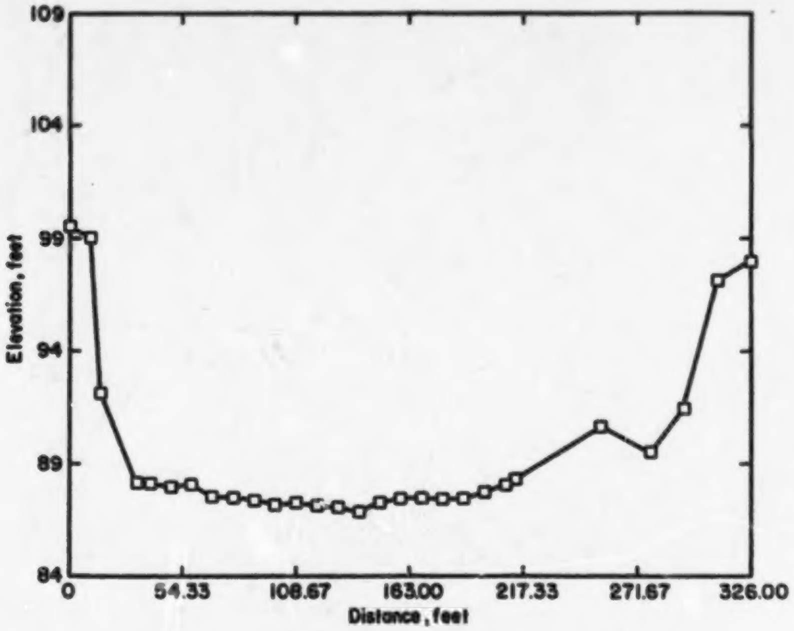


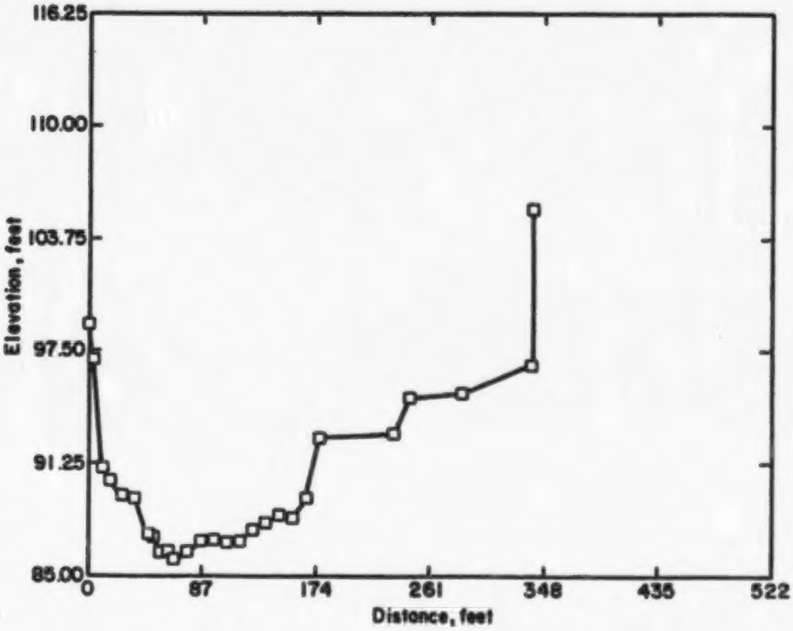
Diagram of a pool riffle habitat used by Colorado squawfish on the Yampa River in Lily Park. (Contours indicate depth in feet, 30 August 1976).



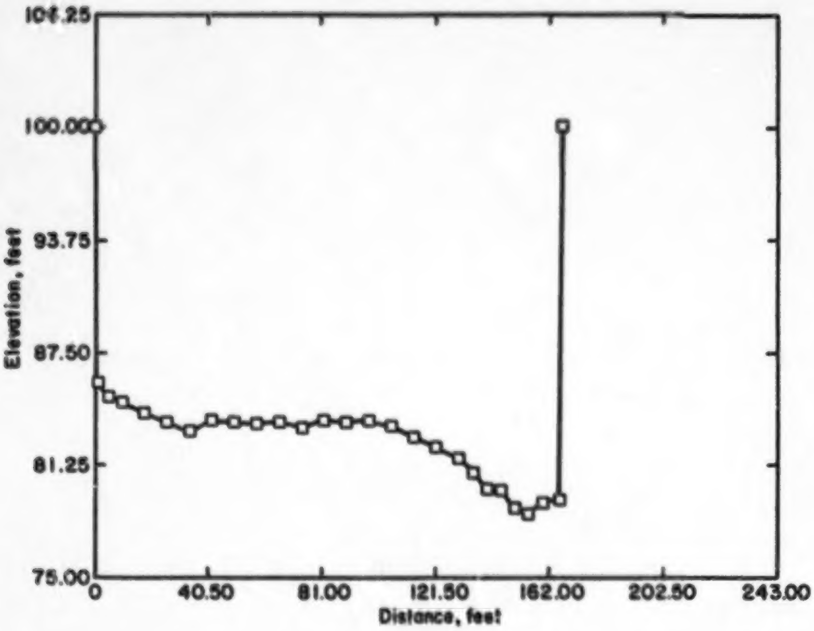
Location of Box Elder and Mantle Ranch reaches.



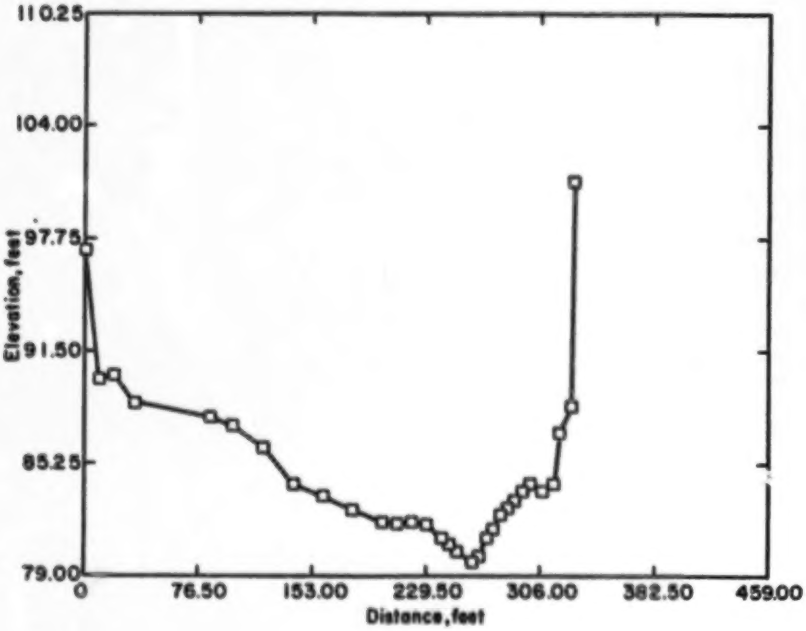
Cross section of Yampa River at Maybell Reach.



Cross section of Yampa River at Lily Park.



Cross section of Yampa River at Dinosaur National Monument near Mantle Ranch.



Cross section of Yampa River at Dinosaur National Monument at Box Elder Reach.

APPENDIX B

CHANNEL CHANGE WORKSHOP: PROBLEM NUMBER 2
POPLAR CREEK, CALIFORNIA

THE QUESTION

The question relative to the Poplar Creek is "What will happen to the morphology of the stream channel as a result of the changes in streamflows and sediment discharge caused by the construction of a reservoir upstream of a reach of stream?" Specifically, for both a short and a long time after construction of the reservoir:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material?
4. What will be the pool riffle sequence?

Poplar Creek is located in a semiarid region of northern California. Poplar is not the actual name of the stream. The problem as presented here has been abstracted from the data for the actual project to the extent it no longer is representative of the concern about the actual project. In order to make it clear the comments on the workshop problem are not necessarily applicable to the actual project, the name of the stream has been changed.

The reach of interest is the reach of the stream from just below Dutch Gulch dam site to the junction with the south fork. Special attention should be given to the reach just downstream of Dry Creek.

CONTENTS[1]

Location of Reach of Interest

Monthly Flows

Cross Sections of Reach of Interest

Peak Flows and Flow Duration Curves

Bed Material

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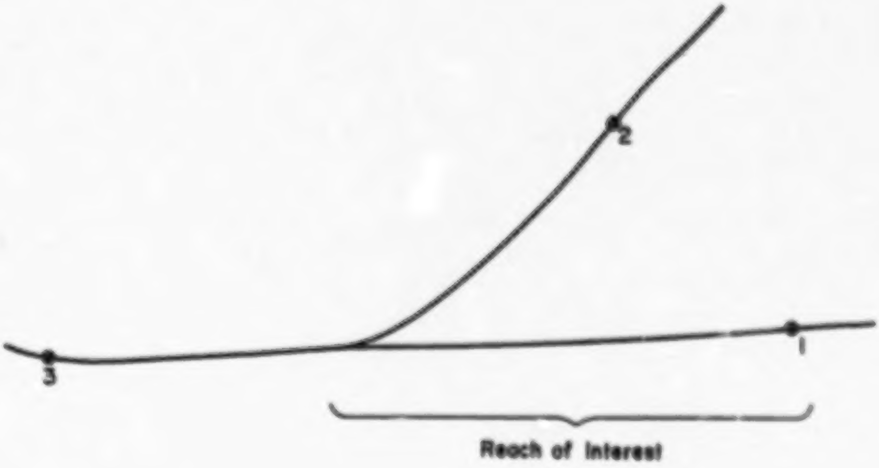
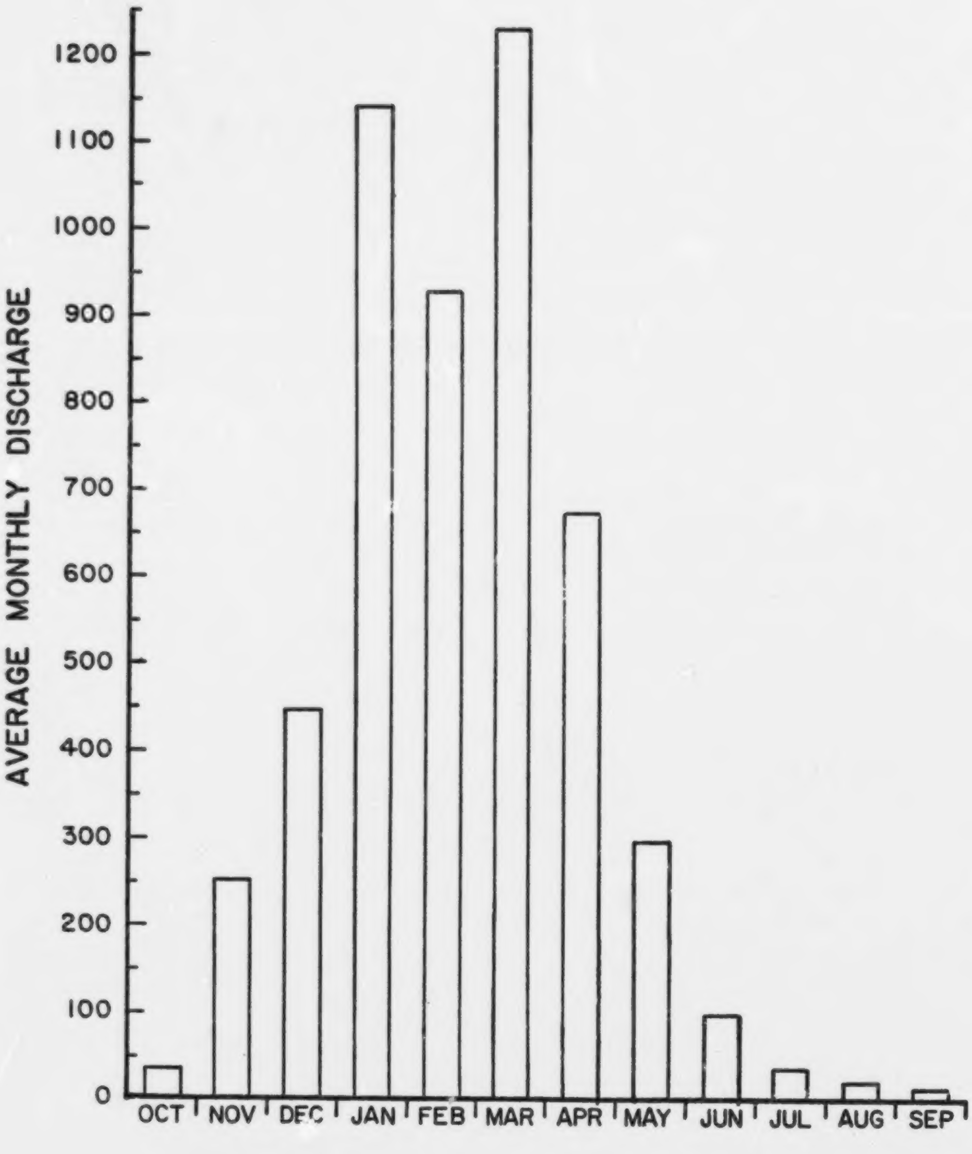
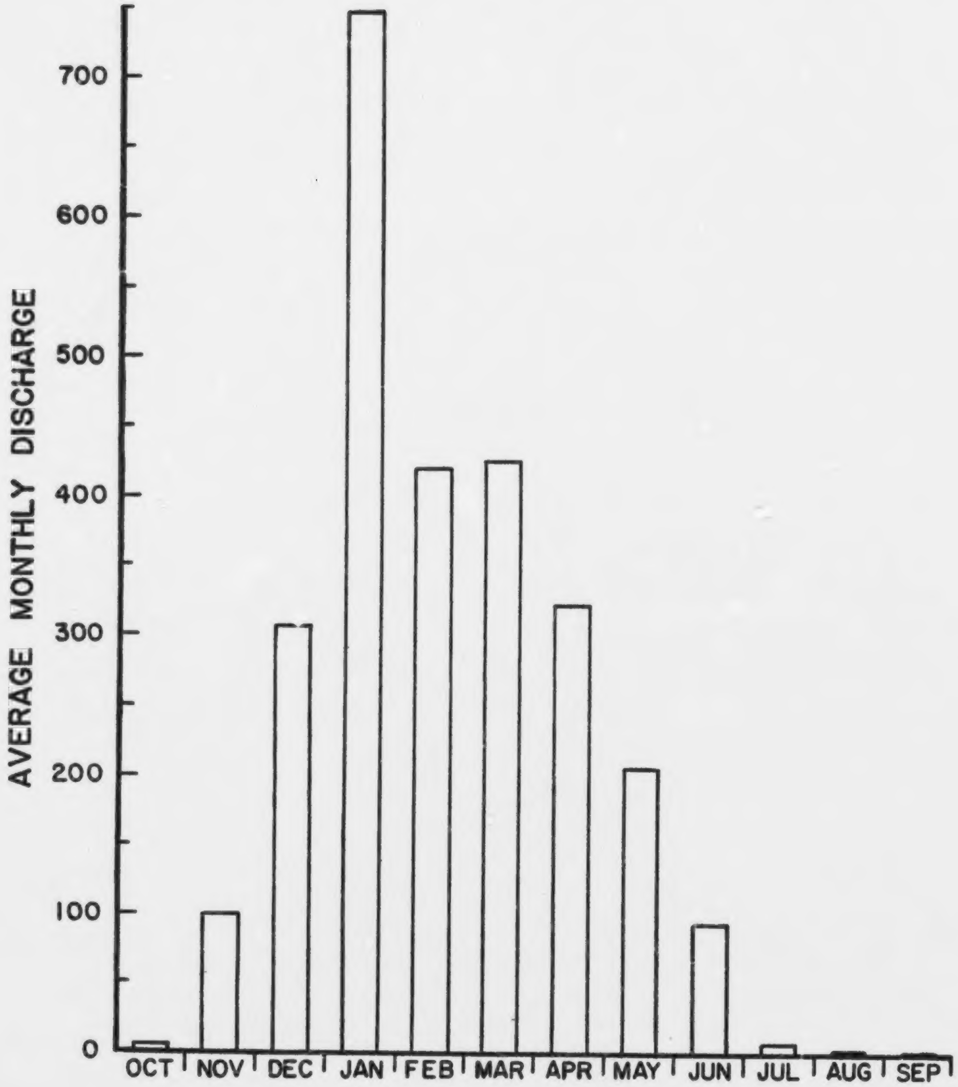
[1] Only selected data are provided in this appendix. Contact the Instream Flow Group of the Fish and Wildlife Service for the complete data set.



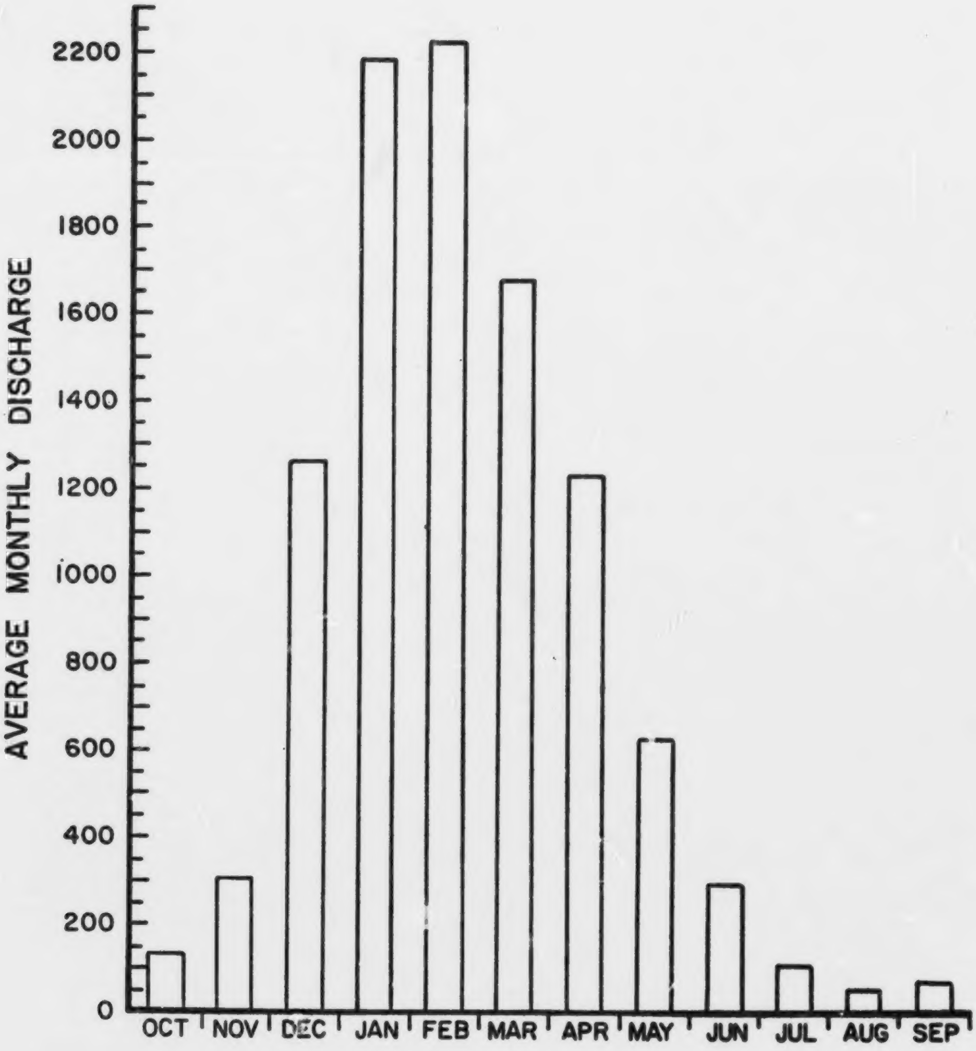
Schematic diagram showing relative location of gaging stations.



Poplar Creek at Station 1 (1972-1978)



Poplar Creek at Station 2 (1963-1978)



Poplar Creek at Station 3 (1941-1979)

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              POPLAR CREEK AT STATION 1
                      CHANNEL CHANGE WORKSHOP PROBLEM
              UNITS OF DISCHARGE ARE  CUSECS

 YEAR       OCT      NOV      DEC      JAN      FEB      MAR      APR      MAY     JUNE     JULY      AUG     SEPT       ANNUAL

 1972       26.      89.     201.     362.     370.     618.     248.     126.      51.      10.       4.       5.        175.
 1973       63.     462.     668.    1871.    1855.    1466.     646.     298.      95.      33.      20.      15.        619.
 1974       65.    1102.    1604.    3155.     795.    1883.    1620.     419.     153.      96.      49.      21.        916.
 1975       32.      61.     158.     215.    1547.    2853.    1000.     590.     197.      73.      28.      21.        560.
 1976       70.     114.     146.      84.     305.     298.     240.      95.      19.       7.      28.      21.        118.
 1977       17.      35.      28.      52.      45.      78.      43.      63.      17.       1.       0.      10.         32.
 1978       17.      99.     699.    2925.    1722.    1811.    1027.     412.     136.      63.      17.      23.        743.
 1979       24.      45.      53.     289.     764.     813.     453.     347.      93.      26.      13.      14.        242.

 AVERAGE    39.     251.     445.    1119.     925.    1227.     660.     294.      95.      38.      20.      16.     426.

 Q/QANN    .780    4.842    8.866   22.317   16.819   24.477   12.729    5.857    1.841     .767     .393     .311  100.000

 COV VAR   .580    1.477    1.207    1.183     .753     .767     .798     .632     .681     .908     .780     .404     .766

 SKEW      .506    2.206    1.711     .878     .254     .496     .766     .146     .205     .590     .704    -.804     .264

 MAXIMUM    70.    1102.    1604.    3155.    1855.    2853.    1620.     590.     197.      96.      49.      23.     916.

 MINIMUM    17.      35.      28.      52.      45.      78.      43.      63.      17.       1.       0.       5.      32.
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STATISTICAL PARAMETERS FOR STATION   11375810     POPLAR CREEK AT STATION 1
  CHANNEL CHANGE WORKSHOP PROBLEM


                              ARTH.                LOG PARAMETERS
                 MONTH       AVERAGE      MEAN   VARIANCE  STD. DEV.    SKEW


        1        OCT          39.14     1.5261     .0667     .2583      .2088
        2        NOV         250.93     2.0870     .2633     .5131     1.1211
        3        DEC         444.59     2.3382     .3541     .5950     -.0609
        4        JAN        1119.11     2.6535     .4743     .6887      .0705
        5        FEB         925.43     2.7734     .2903     .5388    -1.3737
        6        MAR        1227.45     2.9024     .2659     .5157    -1.1586
        7        APR         659.61     2.6368     .2520     .5020    -1.1302
        8        MAY         293.71     2.3622     .1255     .3542     -.6264
        9        JUNE         95.38     1.8464     .1671     .4088     -.7648
       10        JULY         38.45     1.2634     .5628     .7502    -1.4250
       11        AUG          19.70      .7246    2.3693    1.5392    -2.5942
       12        SEPT         16.13     1.1612     .0575     .2397    -1.5946
       13        ANNUAL      425.61     2.4472     .2463     .4963     -.9517


        SAMPLE SIZE    8 YEARS



LOG NORMAL DISTRIBUTION STATION   11375810     POPLAR CREEK AT STATION 1
  CHANNEL CHANGE WORKSHOP PROBLEM


                        YEARS FLOW IS NOT EXCEEDED

     MONTH   9 IN 10   2 IN 3   1 IN 2   1 IN 5  1 IN 10  1 IN 20          Q2 - Q10

     OCT         72.      43.      34.      20.      16.      13.               18.
     NOV        556.     203.     122.      45.      27.      17.               95.
     DEC       1262.     392.     218.      69.      38.      23.              180.
     JAN       3439.     889.     450.     118.      59.      33.              391.
     FEB       2911.    1010.     593.     209.     121.      77.              472.
     MAR       3660.    1329.     799.     294.     174.     113.              624.
     APR       1907.     712.     433.     164.      98.      65.              335.
     MAY        655.     327.     230.     116.      81.      60.              149.
     JUNE       235.     105.      70.      32.      21.      15.               49.
     JULY       168.      38.      18.       4.       2.       1.               16.
     AUG        499.      24.       5.       0.       0.       0.                5.
     SEPT        29.      18.      14.       9.       7.       6.                7.
     ANNUAL    1212.     457.     280.     107.      65.      43.              215.

            Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  11375810
                     POPLAR CREEK AT STATION 1
                          CHANNEL CHANGE WORKSHOP PROBLEM

        DATA SKEW K FACTORS

MONTH       SKEW                                                         RETURN PERIOD

                            10 YRS          6.67 YRS           2 YRS          1.25 YRS          1.11 YRS          1.05 YRS

OCT          .21            1.3002            .4081           -.0316           -.8496           -1.2591           -1.5886
NOV         1.12            1.3408            .2948           -.1766           -.8488           -1.1114           -1.2878
DEC         -.06            1.2653            .4436            .0232           -.8337           -1.2955           -1.6835
JAN          .07            1.2735            .4367            .0120           -.8378           -1.2890           -1.6647
FEB        -1.37            1.0350            .5356            .2289           -.7011           -1.3359           -1.9414
MAR        -1.16            1.0769            .5278            .2012           -.7266           -1.3396           -1.9162
APR        -1.13            1.0706            .5290            .2055           -.7229           -1.3393           -1.9205
MAY         -.63            1.1705            .4993            .1278           -.7826           -1.3352           -1.8337
JUNE        -.76            1.1593            .5036            .1376           -.7761           -1.3371           -1.8457
JULY       -1.43            1.0000            .5404            .2505           -.6788           -1.3300           -1.9592
AUG        -2.59             .7457            .5351            .3685           -.4978           -1.2372           -2.0129
SEPT       -1.59             .9927            .5413            .2548           -.6742           -1.3287           -1.9625
ANNUAL      -.95            1.1179            .5177            .1717           -.7517           -1.3405           -1.8852

FREQUENCY OF FLOWS


                               YEARS FLOW IS NOT EXCEEDED

         MONTH    9 IN 10    2 IN 3    1 IN 2    1 IN 5   1 IN 10   1 IN 20            Q2 - Q10

        OCT           73.       43.       33.       20.       16.       13.                 17.
        NOV          596.      173.       99.       45.       33.       27.                 66.
        DEC         1234.      400.      225.       70.       37.       22.                188.
        JAN         3393.      900.      459.      119.       58.       32.                401.
        FEB         2143.     1153.      788.      249.      113.       53.                675.
        MAR         2869.     1495.     1014.      337.      163.       82.                852.
        APR         1494.      799.      549.      188.       92.       47.                457.
        MAY          598.      346.      256.      122.       77.       52.                178.
        JUNE         209.      113.       80.       34.       20.       12.                 60.
        JULY         103.       47.       28.        6.        2.        1.                 26.
        AUG           75.       35.       20.        1.        0.        0.                 20.
        SEPT          25.       20.       17.       10.        7.        5.                 10.
        ANNUAL      1005.      506.      341.      119.       61.       32.                280.

                   Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
```

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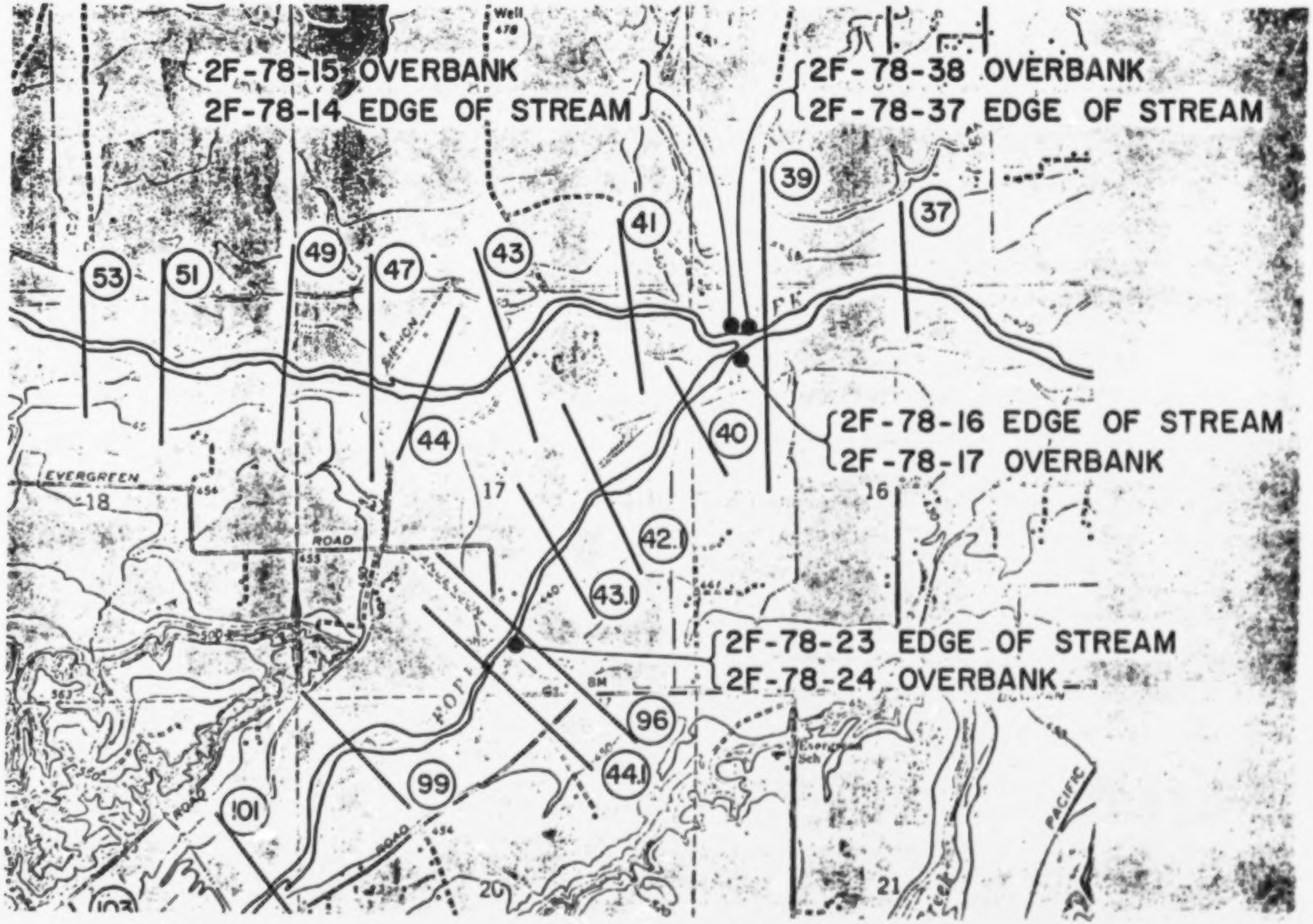
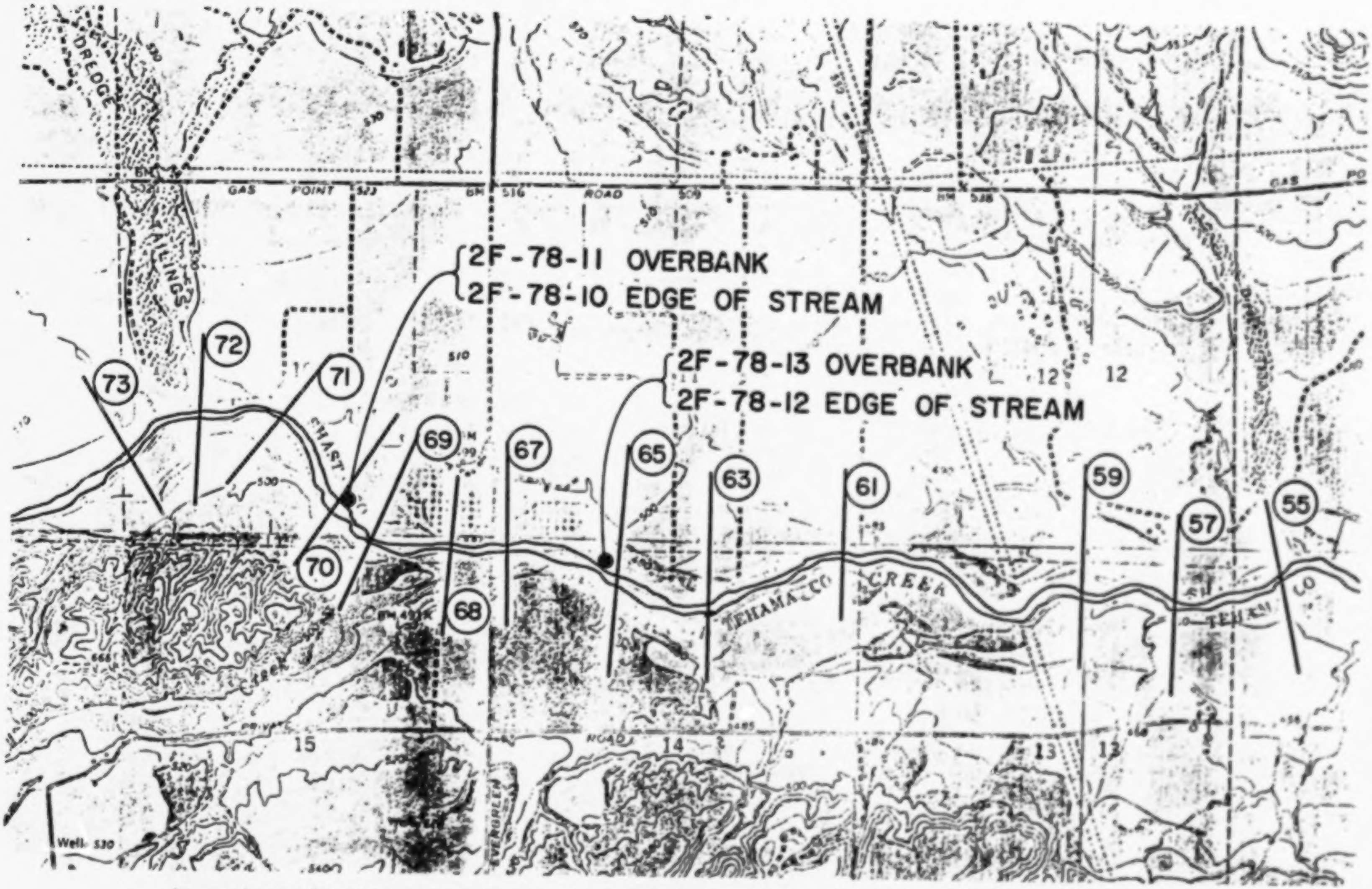
CROSS SECTIONS OF REACH OF INTEREST



Approximate location of cross sections and soil samples (scale 1:24,000 approx., April 1980).



Approximate location of cross sections and soil samples (scale 1:24,000 approx., April 1980, continued).

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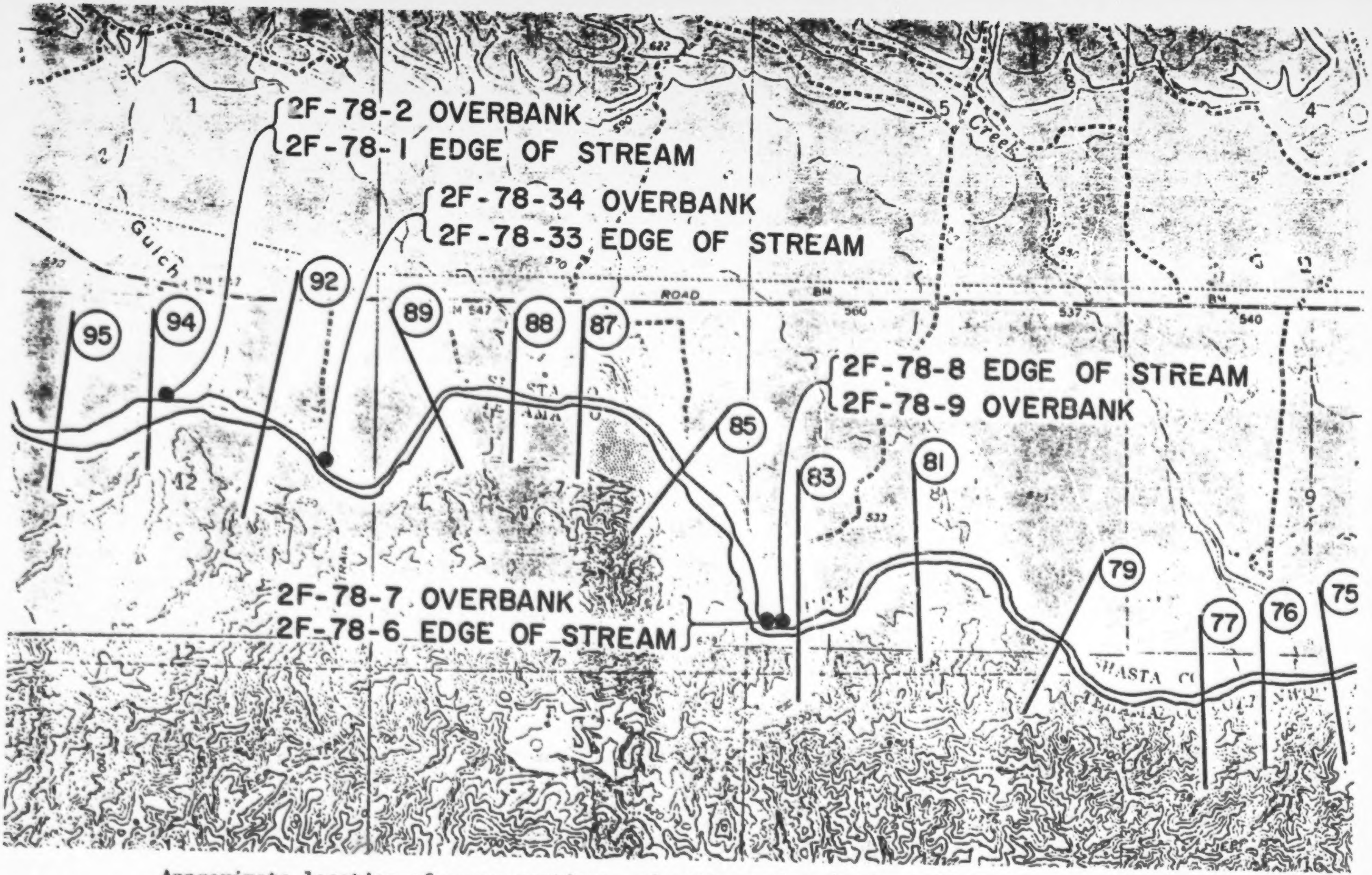
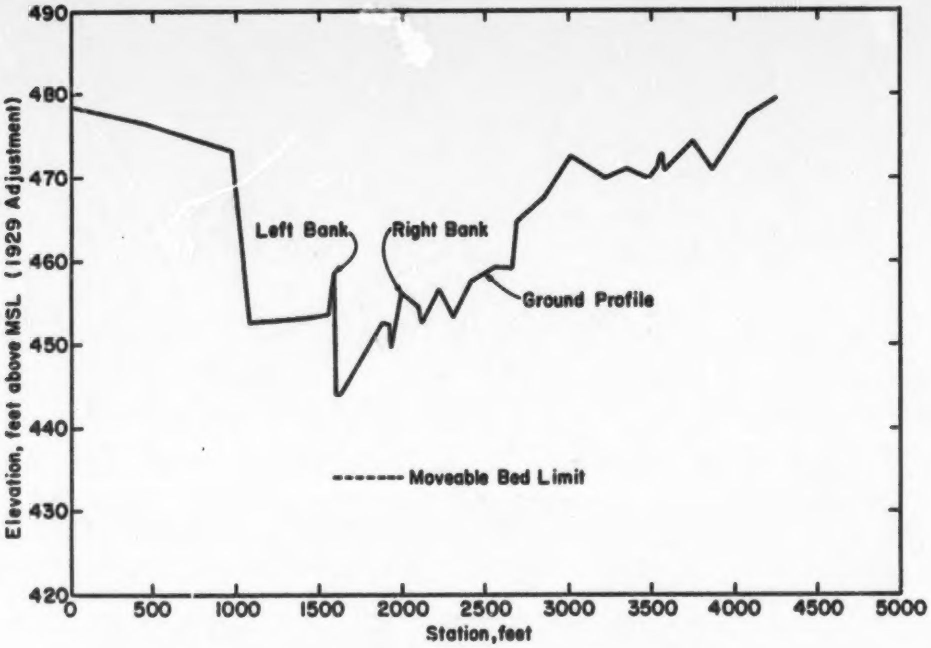
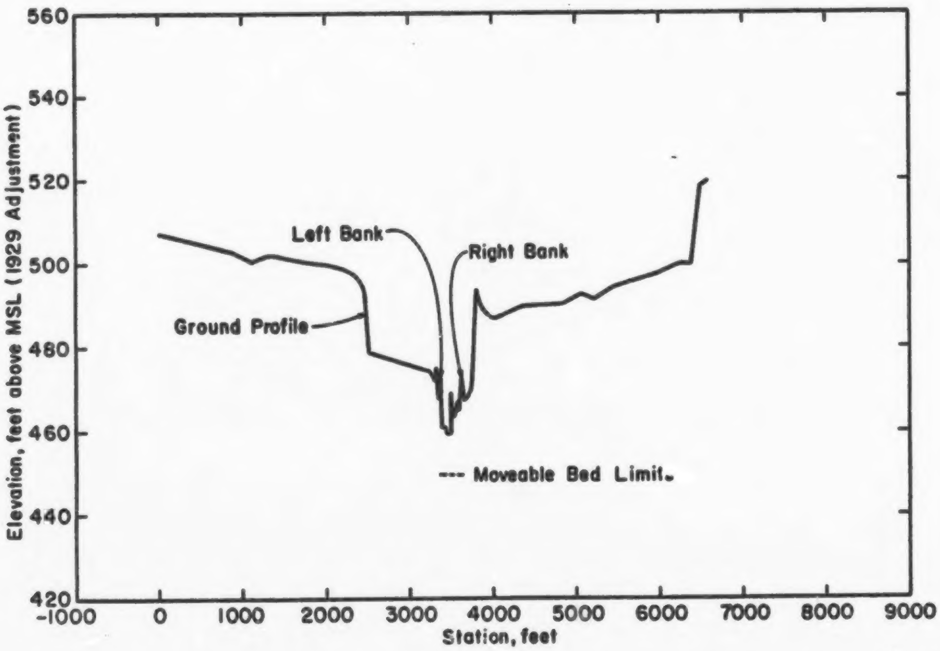


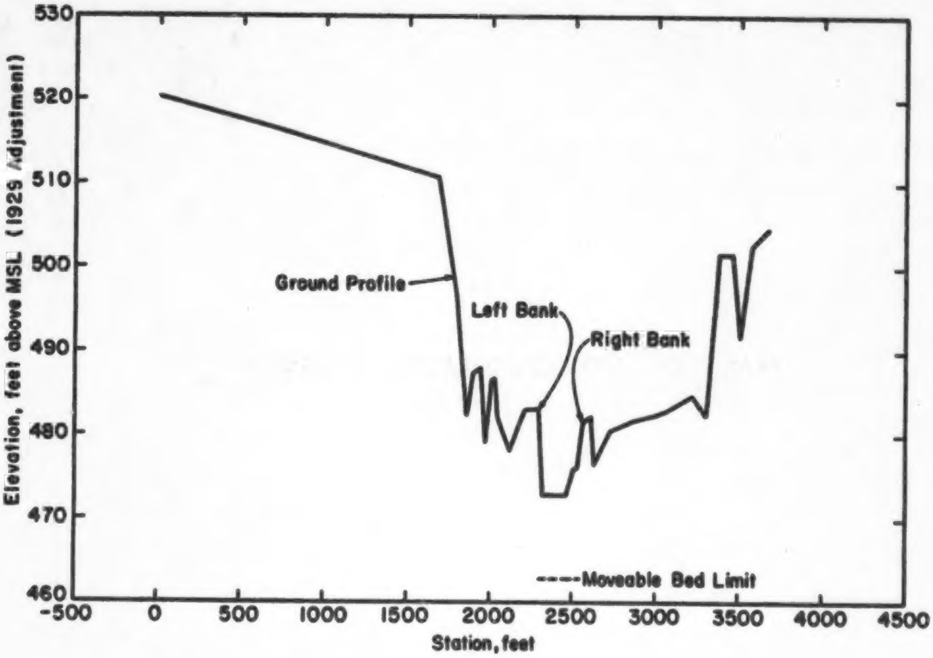
Approximate location of cross sections and soil samples (scale 1:24,000 approx., April 1980, continued).



Station compared to elevation for Section 57.



Station compared to elevation for Section 67.



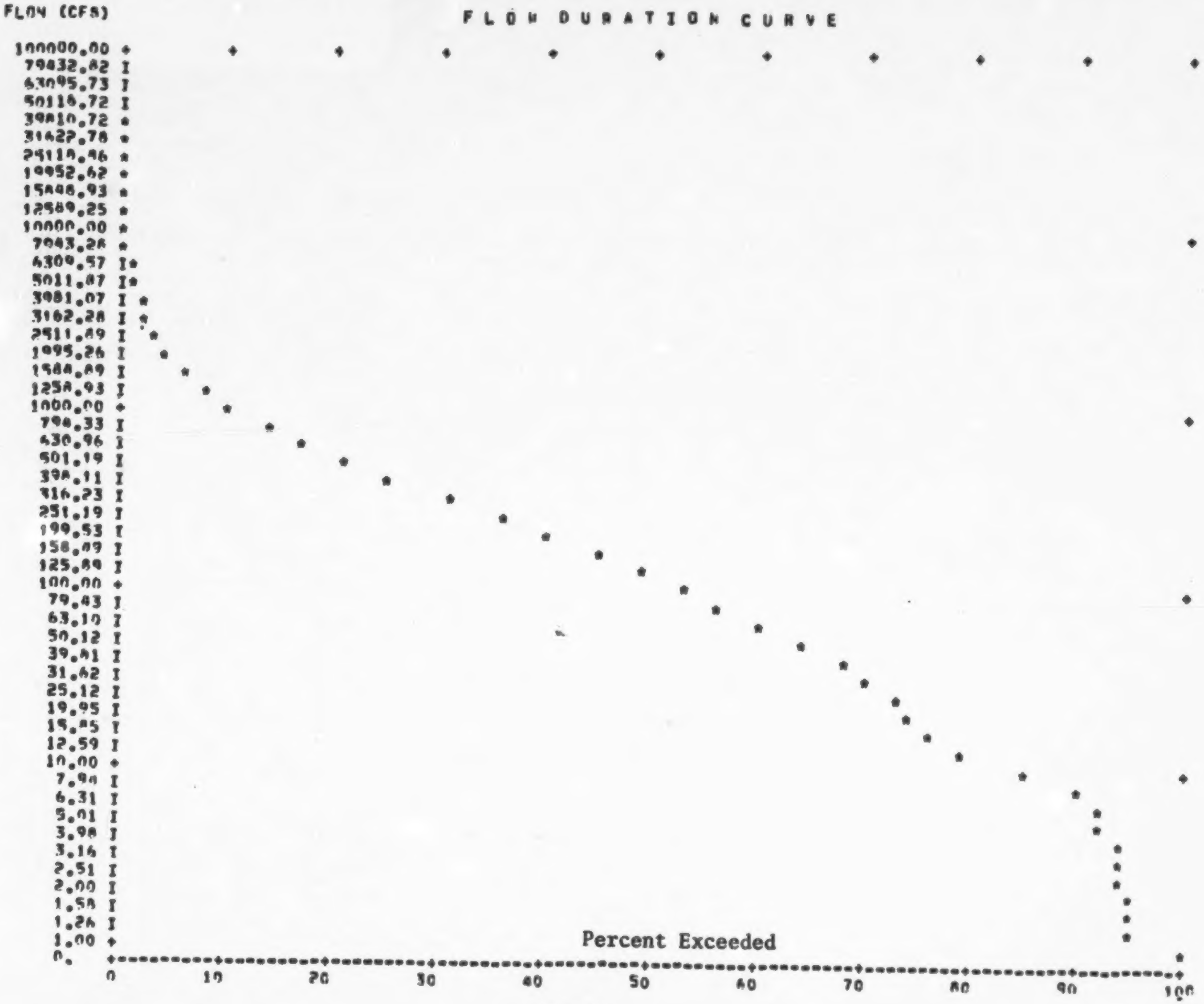
Station compared to elevation for Section 72.

PEAK FLOWS AND FLOW DURATION CURVES

NATURAL CONDITIONS

JUST ABOVE REACH OF INTEREST

JOB NUMBER 1



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*********************************************************************************

                                                 NATURAL CONDITIONS
JOB NUMBER    1
*********************************************************************************

STREAM FLOW DATA INPUT

FL CARD       FTYPE      FMTYPE       PUNCH
               FILE         STD          NO

FM CARD        TERM      ITVALS       NVALS
            -999.00     1000000           0

 UNADJUSTED FLOW DURATION CURVE -- FLOWS IN CFS AND EXCEEDENCE PROBABILITIES AS DECIMAL FRACTIONS

PE           1.0000       .9467       .9467       .9467       .9416       .9365       .9365       .9238       .9162       .9035
QQ             1.00        1.26        1.58        2.00        2.51        3.16        3.98        5.01        6.31        7.94

PE            .8451       .7918       .7639       .7436       .7257       .6979       .6777       .6408       .6033       .5642
QQ            10.00       12.59       15.85       19.95       25.12       31.62       39.81       50.12       63.10       79.43

PE            .5271       .4931       .4472       .3975       .3553       .3110       .2504       .2098       .1750       .1362
QQ           100.00      125.89      158.49      199.53      251.19      316.23      398.11      501.19      630.96      794.33

PE            .1008       .0754       .0561       .0418       .0306       .0216       .0150       .0097       .0059       .0031
QQ          1000.00     1258.93     1584.89     1995.26     2511.89     3162.28     3981.07     5011.87     6309.57     7943.28

PE            .0015       .0006       .0003       .0002       .0001       .0000          0.
QQ         10000.00    12589.25    15848.93    19952.62    25118.86    31622.78    39810.72
```

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************************************************************************************************

JOB NUMBER    1                  PROJECT CONDITIONS                      JUST ABOVE REACH OF INTEREST
************************************************************************************************
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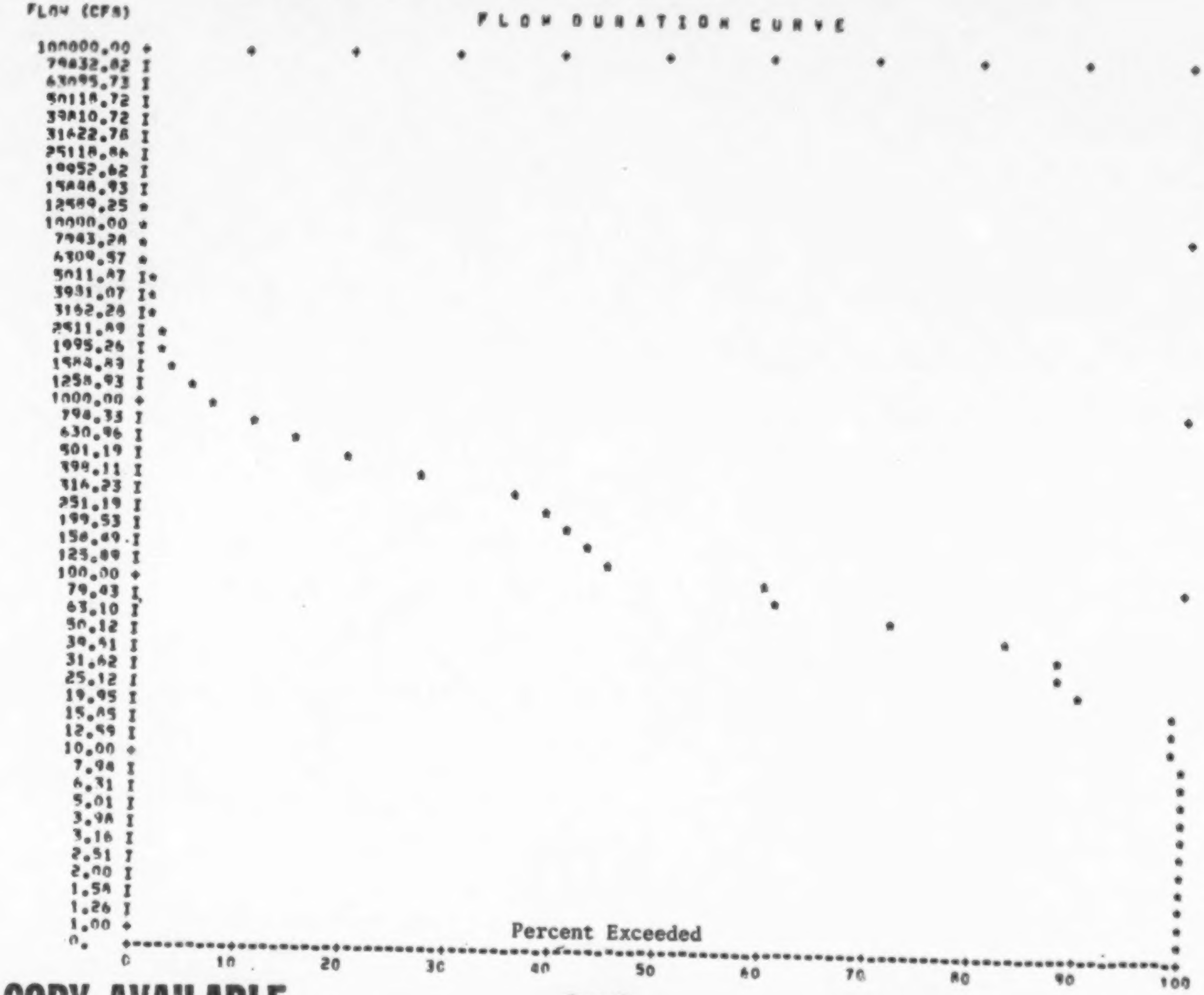

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309

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*****************************************************************************************

JOB NUMBER     1                       PROJECT CONDITIONS
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STREAM FLOW DATA INPUT

FL CARD          FTYPE        FMTYPE          PUNCH
                  FILE           STD             NO

FM CARD           TERM        ITVALS          NVALS
               -999.00       1000000              0

 UNADJUSTED FLOW DURATION CURVE -- FLOWS IN CFS AND EXCEEDENCE PROBABILITIES AS DECIMAL FRACTIONS

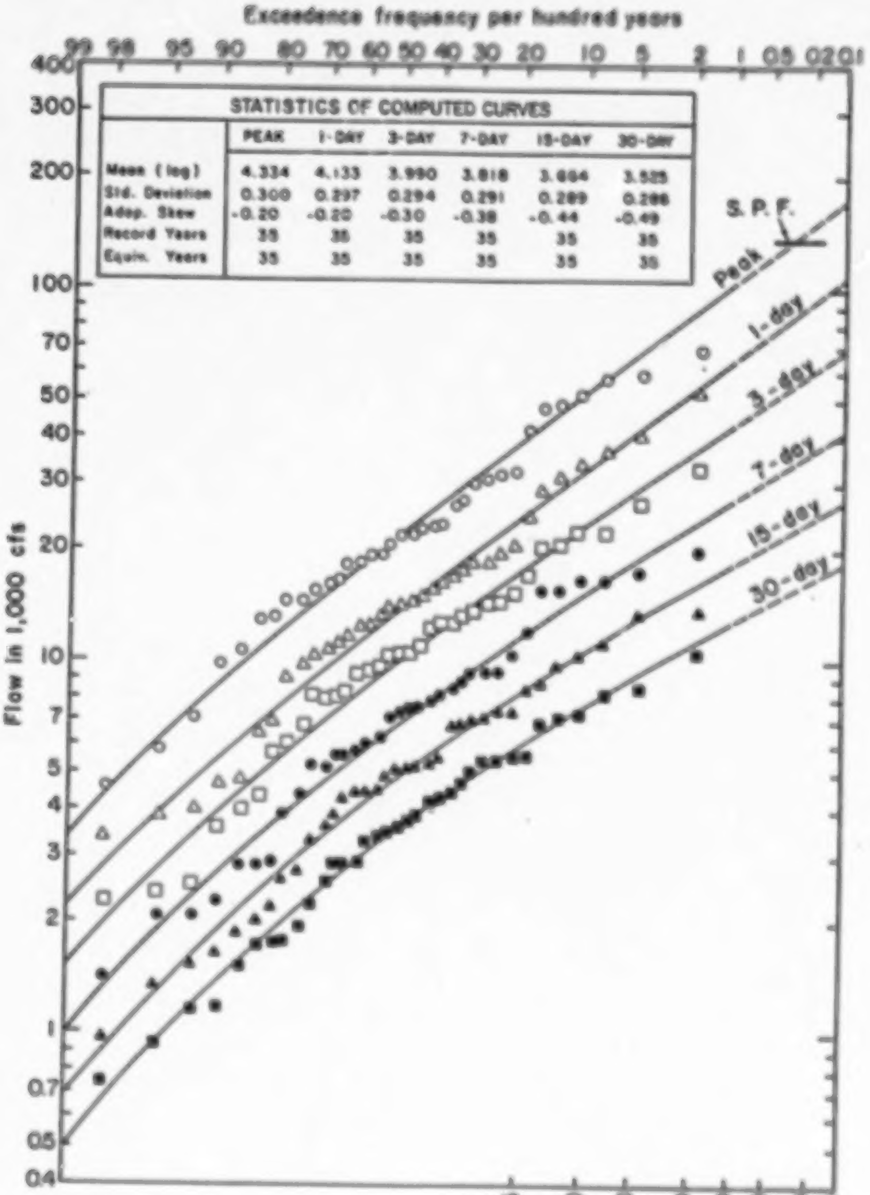
PE              1.0000        1.0000        1.0000        1.0000         .9975         .9975         .9975         .9975         .9975         .9975
QQ                1.00          1.26          1.58          2.00          2.51          3.16          3.98          5.01          6.31          7.94

PE               .9975         .9902         .9902         .9902         .8970         .8847         .8823         .8295         .7233         .6114
QQ               10.00         12.59         15.85         19.95         25.12         31.62         39.81         50.12         63.10         79.43

PE               .6032         .4534         .4349         .4123         .3941         .3567         .2672         .1994         .1538         .1093
QQ              100.00        125.89        158.49        199.53        251.19        316.23        398.11        501.19        630.96        794.33

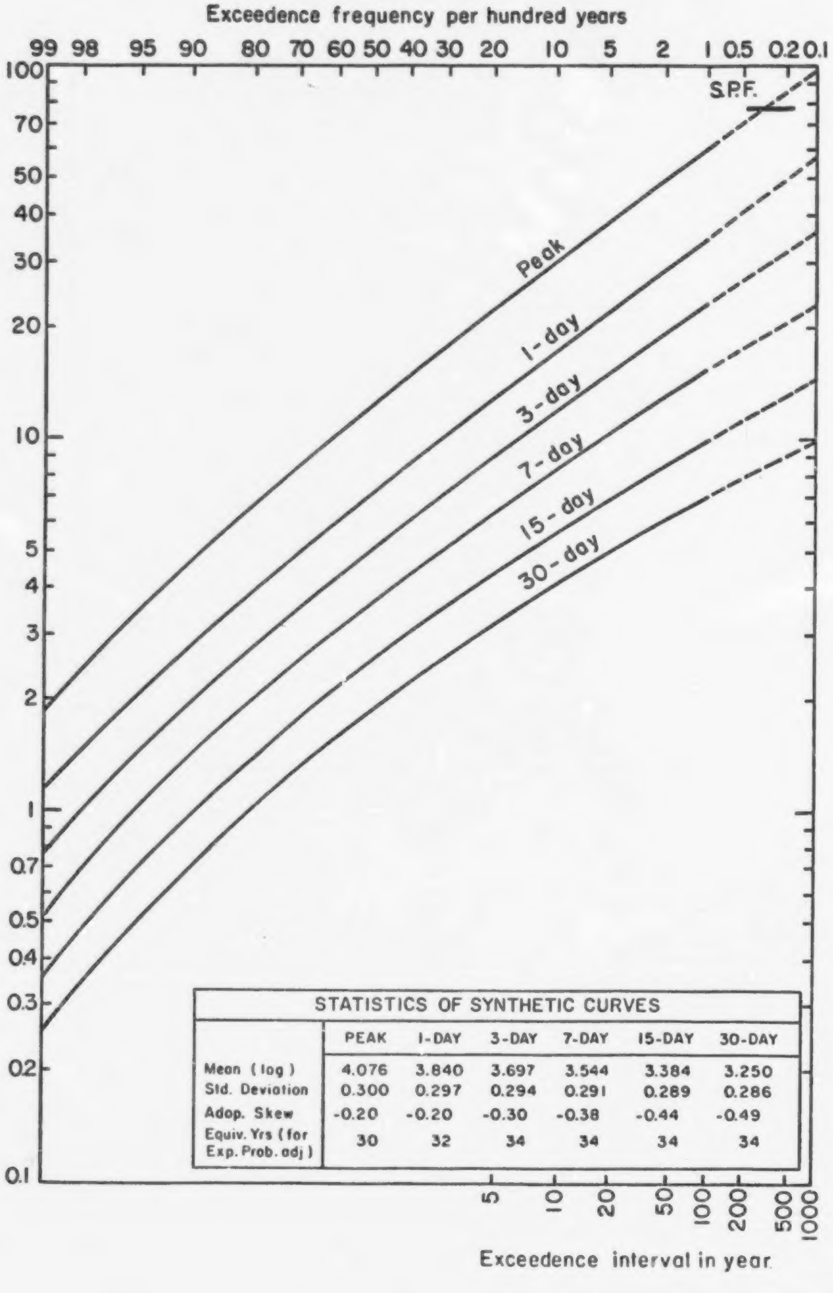
PE               .0654         .0457         .0350         .0248         .0190         .0145         .0108         .0075         .0049         .0032
QQ             1000.00       1258.93       1584.89       1995.26       2511.89       3162.28       3981.07       5011.87       6309.57       7943.28

PE               .0007        0.
QQ            10000.00      12589.25
```



STATISTICS OF COMPUTED CURVES						
	PEAK	1-DAY	3-DAY	7-DAY	15-DAY	30-DAY
Mean (log)	4.334	4.133	3.990	3.818	3.664	3.525
Std. Deviation	0.300	0.297	0.294	0.291	0.289	0.286
Adop. Skew	-0.20	-0.20	-0.30	-0.38	-0.44	-0.49
Record Years	35	35	35	35	35	35
Equiv. Years	35	35	35	35	35	35

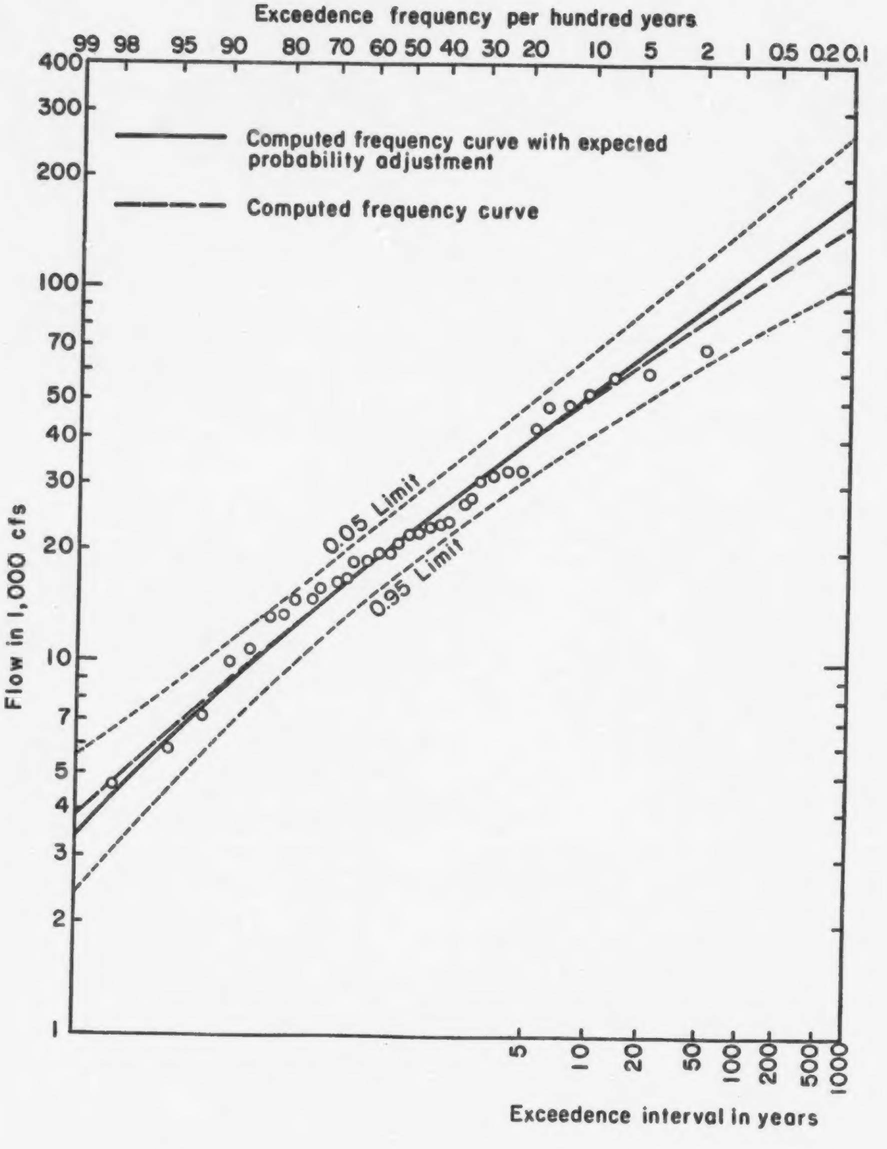
Flow frequency Poplar Creek near Site 1 (Corps of Engineers, April 1977).
Drainage area: 927 sq. mi. Period of record: 1941-1975.



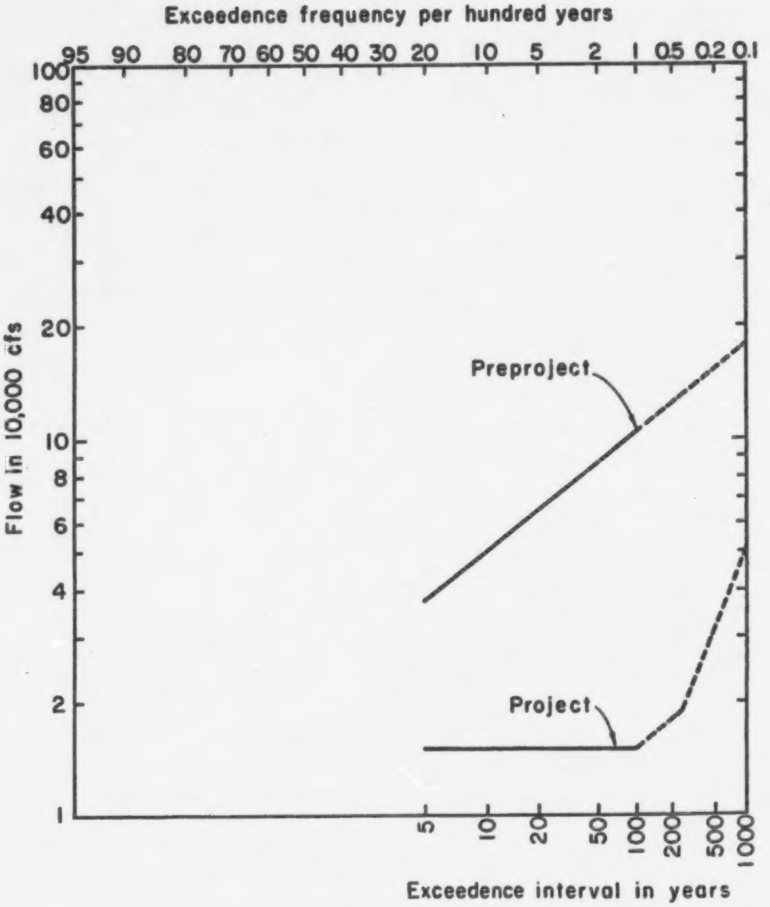
STATISTICS OF SYNTHETIC CURVES

	PEAK	1-DAY	3-DAY	7-DAY	15-DAY	30-DAY
Mean (log)	4.076	3.840	3.697	3.544	3.384	3.250
Std. Deviation	0.300	0.297	0.294	0.291	0.289	0.286
Adop. Skew	-0.20	-0.20	-0.30	-0.38	-0.44	-0.49
Equiv. Yrs (for Exp. Prob. adj)	30	32	34	34	34	34

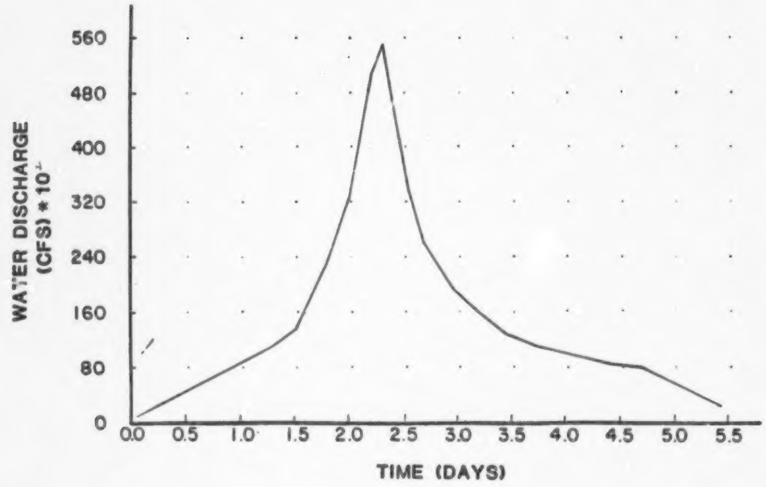
Flow frequency Poplar Creek at dam site above Site 1 (Corps of Engineers, April 1977).
Drainage area: 394.2 sq. mi.



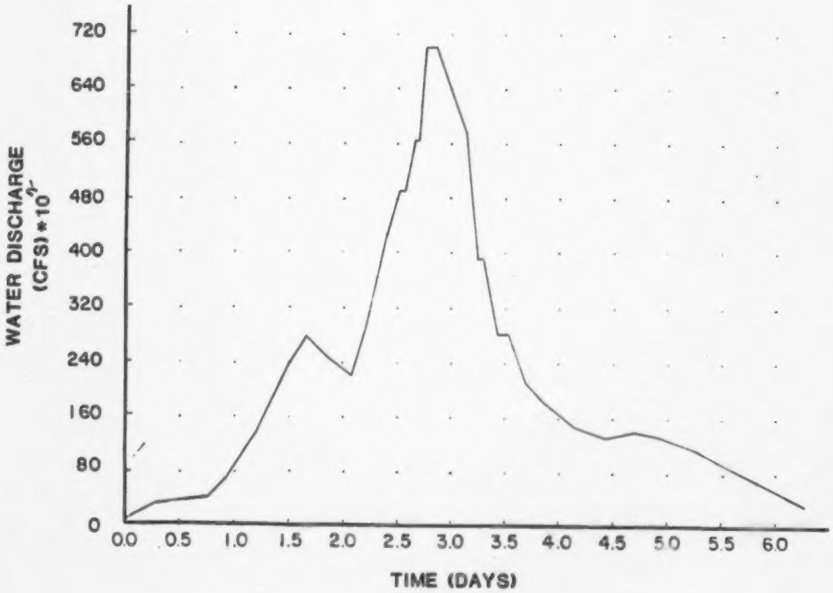
Peak flow frequency Poplar Creek at Site 3 (Corps of Engineers, April 1977).
Drainage area: 927.0 sq. mi. Period of record: 1941-1975.



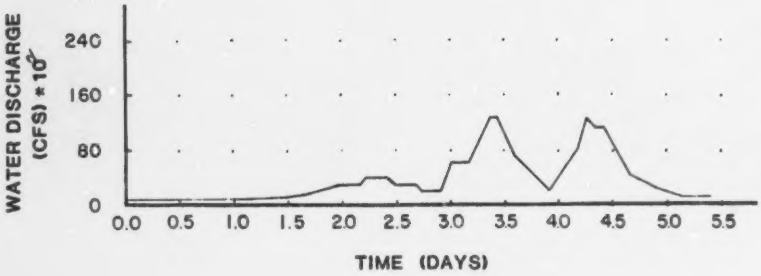
Peak flow frequency Poplar Creek near Site 1 for preproject and postproject conditions (Corps of Engineers, July 1969).



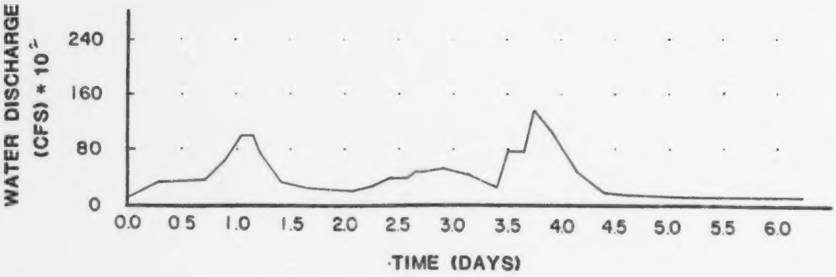
Poplar Creek preproject flood (1970).



Poplar Creek preproject flood (1974).



Poplar Creek project flood (1970).



Poplar Creek project flood (1974).

BED MATERIAL

Total Depth of Hole: 1.0 ft each
Date Started: 18 July 78
Date Completed: 19 July 78

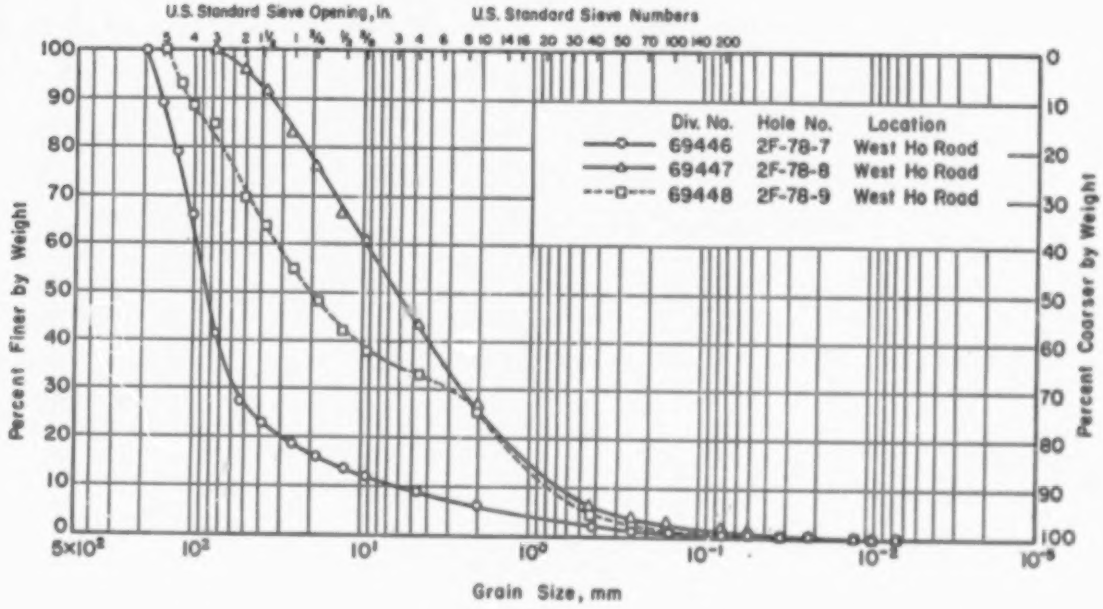
Size and Type of Bit: Shovel
Manufacturer's Designation of Drill:
Ames Middleweight #2

DEPTH	CLASSIFICATION OF MATERIALS	TEST HOLE NUMBERS
0.0 0.5 1.0	sandy gravel, 75% rounded gravel, 25% fine to coarse subangular sand, to 4" maximum.	2F-78-8
0.0 0.5 1.0	sandy gravel with scattered cobbles. 75% rounded gravel, 25% fine to coarse subangular sand, to 5" maximum.	2F-78-9
0.0 0.5 1.0	cobbly sandy gravel, 60% rounded gravel, 30% fine to coarse angular sand, 10% rounded cobbles.	2F-78-10
0.0 0.5 1.0	sandy gravel with cobbles, 90% rounded cobbles 10% fine-coarse, angular sand, cobbles to 6" maximum.	2F-78-11
0.0 0.5 1.0	sandy, gravelly cobbles, 40% rounded cobbles, 30% rounded gravels, 30% fine to coarse angular sand.	2F-78-12
0.0 0.5 1.0	sandy gravel with scattered cobbles, 90% rounded gravels, 10% fine-coarse angular sand, Cobbles to maximum 6".	2F-78-13
	19 July 78	
0.0 0.5 1.0	Sandy gravel, 65% gravel, 35% sand, Gravel maximum dimension 5" but would go through 3" square.	2F-78-14

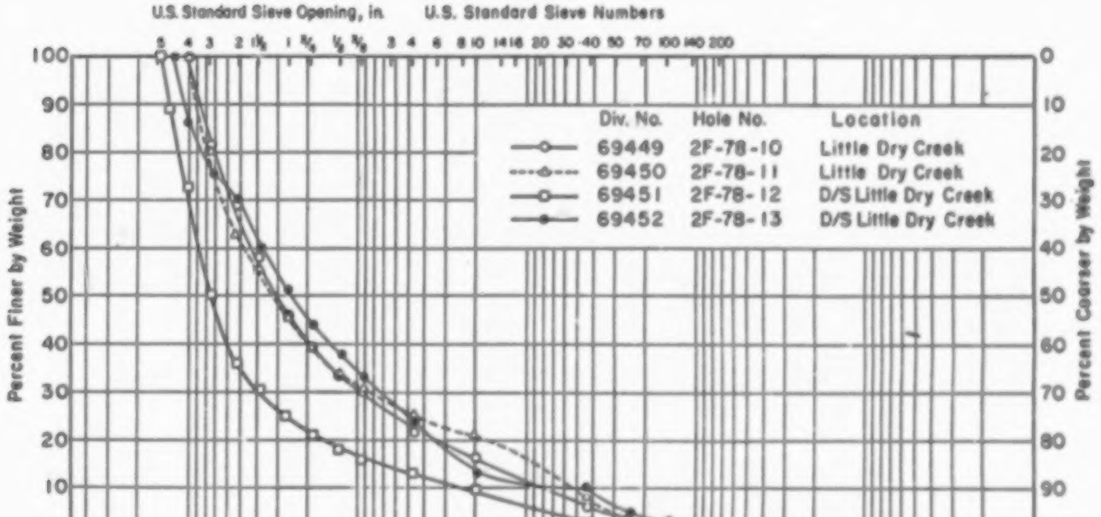
Drilling log for Poplar Creek sedimentation study

DIVISION SERIAL NO.	HOLE NO.	CO-ORD OR STA.	MECHANICAL ANALYSIS--% FINER										
			Gravel				Sand				Fines		
			6/3	2/1 1/2	1/3/4	1/2/3/8	No. 4	No. 10	No. 40	No. 80	No. 200		
69440	2F-78-1	Dutch Gulch	100	98/97	94/91	88/84	74	57	19	9	1		2 Sacks
69441	2F-78-2	Dutch Gulch		100/96	94/86	76/67	52	37	8	3	0.5		2 Sacks
69442	2F-78-3	Dutch Gulch Bridge	82/58	45/40	32/27	22/20	16	9	3	2	1	42% Cobbles	4 Sacks
69443	2F-78-4	Dutch Gulch Bridge	94/80	74/72	68/66	64/63	62	50	8	5	4	20% Cobbles	3 Sacks
69444	2F-78-5	Dutch Gulch Bridge	100	97/91	79/73	65/61	52	35	3	2	1		2 Sacks
69445	2F-78-6	West Ho Road	91/64	55/49	41/35	30/27	21	16	5	4	2	36% Cobbles	4 Sacks
69446	2F-78-7	West Ho Road	89/41	27/23	18/15	13/12	9	6	2	2	1	59% Cobbles	4 Sacks
69447	2F-78-8	West Ho Road	100	97/93	83/76	68/61	43	27	8	5	1		2 Sacks
69448	2F-78-9	West Ho Road	100/85	71/65	56/48	42/39	33	26	5	3	1	15% Cobbles	2 Sacks
69449	2F-78-10	Little Dry Cr.	100/82	69/58	46/40	34/30	22	17	6	4	1	18% Cobbles	3 Sacks
69450	2F-78-11	Little Dry Cr.	100/80	63/56	46/40	35/31	26	22	9	4	1	20% Cobbles	4 Sacks
69451	2F-78-12	D/S Little Dry Cr.	100/50	36/31	25/21	18/16	13	10	3	2	1	50% Cobbles	4 Sacks
69452	2F-78-13	D/S Little Dry Cr.	100/75	70/60	51/45	38/33	23	13	10	5	1	25% Cobbles	2 Sacks
69453	2F-78-14	Confluence of N&S Forks	100/98	90/79	64/52	43/37	29	21	7	3	0.5	2% Cobbles	2 Sacks
69454	2F-78-15	Confluence of N&S Forks	80/41	31/28	23/19	16/14	10	7	3	1	0	59% Cobbles	4 Sacks
69455	2F-78-16	Confluence of N&S Forks		100/93	82/72	62/54	39	29	9	6	2		2 Sacks
69456	2F-78-17	Confluence of N&S Forks	100/78	73/70	62/56	48/42	27	14	6	3	2	22% Cobbles	2 Sacks

Soil test result summary for Poplar Creek, August 1978.



Sedimentation study (Poplar Creek, 2F-78-7 to 2f-78-9).



Sedimentation study (Poplar Creek, 2f-78-10 to 2F-78-13).

APPENDIX C

CHANNEL CHANGE WORKSHOP: PROBLEM NUMBER 3
ELK RIVER, KANSAS

THE QUESTION

The question relative to the Elk River is "What will happen to the morphology of the stream channel as a result of the changes in streamflows and sediment discharge caused by the construction of a number of small flood retaining reservoirs upstream of a reach of stream?" Specifically, for both a short and long time after construction of the reservoirs:

1. What will be the meander pattern?
2. What will be the configuration of the channel?
3. What will be the substrate material?
4. What will be the pool riffle sequence?

Elk River is located in southeastern Kansas in an area with subhumid climate. The materials in this appendix describe the basin and present the data on both pre and post project conditions. The soils data are for Chautauqua County, the county immediately below Elk County. The Elk River is located in the southern part of Elk County.

The reach of interest is the reach of the river near Longton, Kansas; specifically between Cross Sections 11-15 and 3-3 as shown on the enclosed plan.

Most of the data enclosed were obtained from the Kansas State Office of the Soil Conservation Service. Other data were obtained from U.S. Geological Survey reports. The geology data are from a Kansas Geological Survey report.

CONTENTS

General Location of Elk River, Kansas

General Plan - Elk River Watershed Joint District No. 47 (Partial)[1]

Soil Survey of Chautauqua County, Kansas (Partial)[1]

Geology, Mineral Resources and Groundwater Resources of Elk County, Kansas (Partial)[1]

Plan, Profile, and Cross Section Diagrams for the Longton Reach of the Elk River, Kansas[2]

Monthly Streamflow Data for the Elk River Basin, Kansas

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[1] These items are not included in the information supplied in this appendix. The following bibliography lists the sources of these data.
[2] Only part of the diagrams included in the original data set are supplied and included in this appendix. The others may be reviewed at the office of the Instream Flow Group in Fort Collins, Colorado.

Elk River Bed Material

Elk River Sediment Yield

Frequency - Runoff Curve

Elk River Hydrographs and Elevation vs. Discharge Data

- Unit Discharge Hydrograph
- Unit Discharge Hydrograph Table
- Elevation - Discharge Plots
- Elevation - Discharge Table

USGS MAPS SHOWING THE ELK RIVER AREA INCLUDED IN THE ORIGINAL DATA SETS

1:250,000
  Joplin, Missouri; Kansas
  Wichita, Kansas

1:24,000
  Longton NW, Kansas
  Elk Falls, Kansas
  Longton, Kansas
  Oak Valley, Kansas

BIBLIOGRAPHY OF REPORTS USED IN DATA SET

General Plan - Elk River Watershed Joint District No. 47, Kansas State Office, Soil Conservation Service, USDA, Salina, Kansas, January 1967.

Bell, E. L., and H. T. Rowland. Soil Survey of Chautauqua County, Kansas, Soil Conservation Service, USDA, Washington, D.C., October 1976.

Verville, G. J. R., Kulstad, N. Plummer, W. H. Schoewa, E. D. Goebel, and C. K. Bayhe. Geology, Mineral Resources, and Groundwater Resources of Elk County, Kansas, State Geological Survey of Kansas (Volume 14), Lawrence, Kansas, July 1958.

PLAN, PROFILE, AND CROSS SECTION DIAGRAMS FOR THE LONGTON REACH OF THE ELK RIVER, KANSAS

The following diagrams illustrate the reach of the Elk River for which an estimate of the future form of the channel and substrate is needed. The reach is between Sections 3-3 and 11-15. Only a few of the diagrams made available to the participants are included in this appendix.

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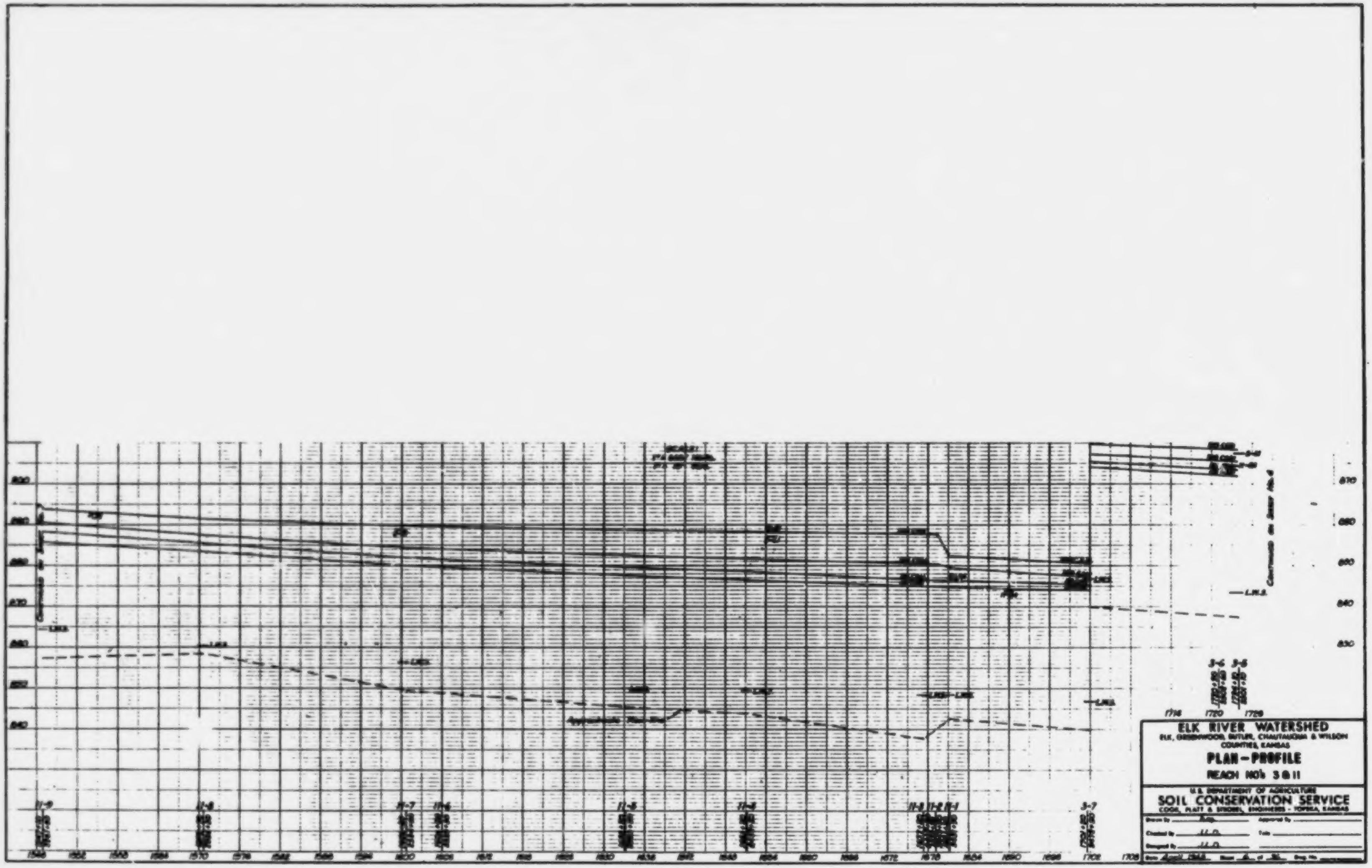

ELK RIVER WATERSHED
ELK, GREENWOOD, BUTLER, CHAUTAUQUA & WILSON
COUNTIES, KANSAS

PLAN – PROFILE

REACH NO's 3 & 11

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
COOK, FLATT & STROBEL, ENGINEERS - TOPEKA, KANSAS

Drawn by Bdg. Approved by
Checked by H.D. Title
Designed by H.D.
Date April [illegible] Sheet 6 of 16 Dwg. No.

324

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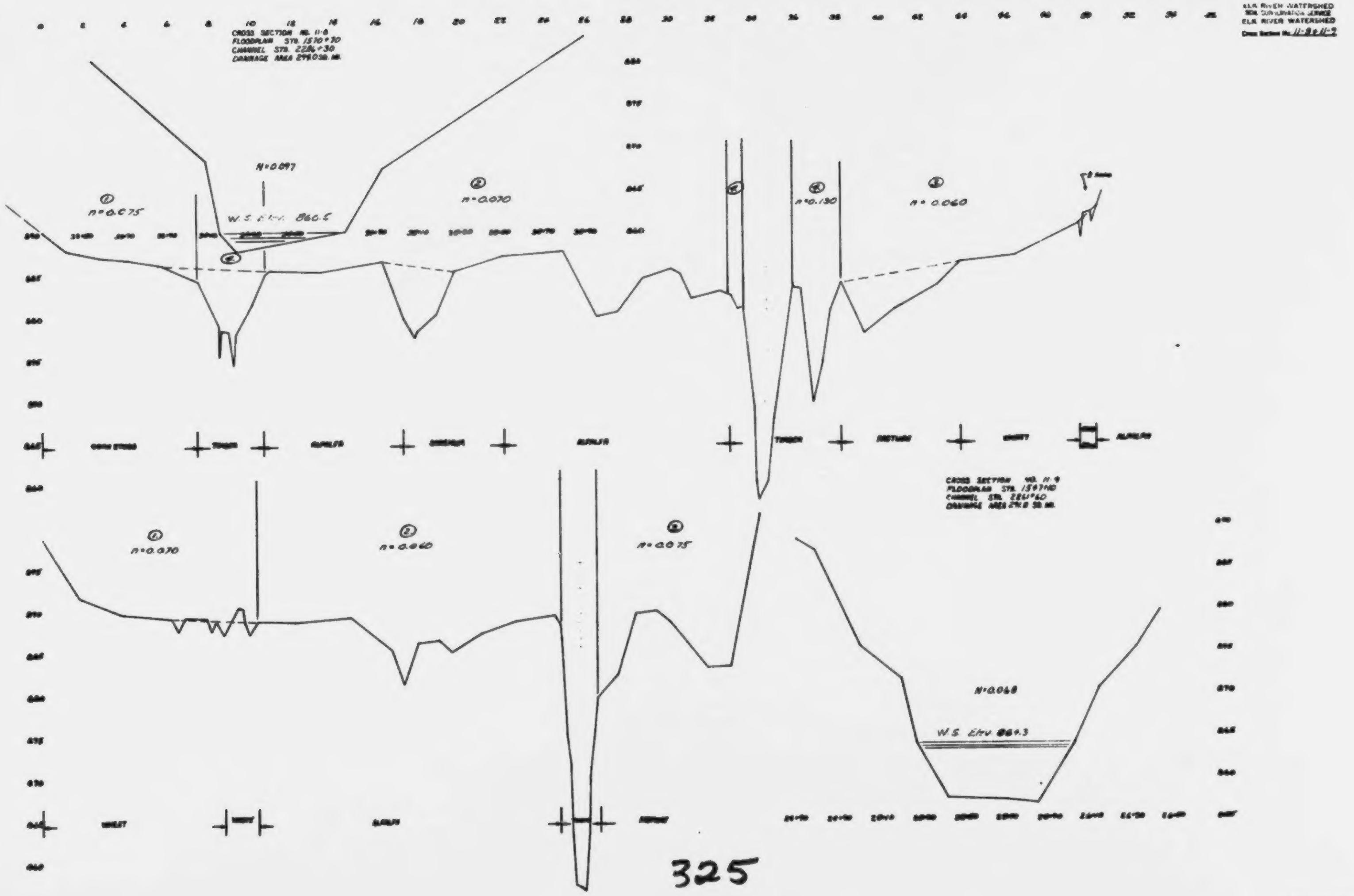


COOK, FLATT & STROBEL
ENGINEERS - TOPEKA, KANSAS

DIAGRAMS INCLUDED IN ORIGINAL DATA SETS

Plan-Profile Reach Numbers 3 and 11

Plan-Profile Reach Numbers 11 and 13

Cross Section 3-3
3-4
3-5
11-1 and 11-2
11-3 and 11-4
11-5 and 11-7
11-8 and 11-9
11-14 and 11-15

MONTHLY STREAMFLOW DATA FOR THE ELK RIVER BASIN, KANSAS

Drainage Areas

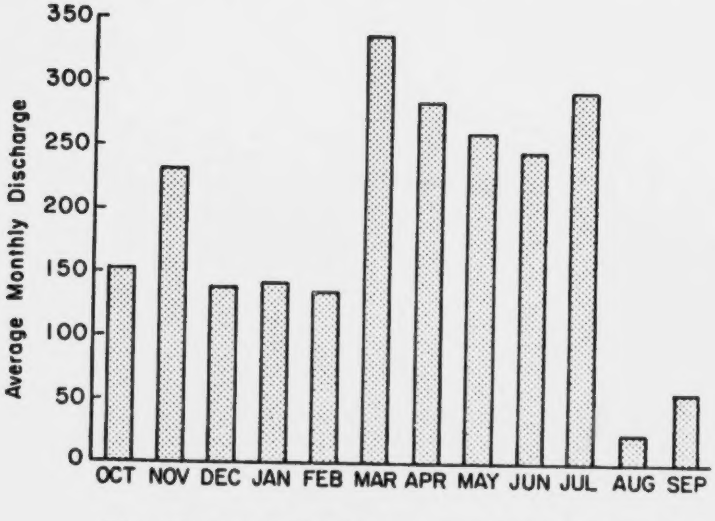
Study Reach:

Head of Reach: 285.7 square miles
Bottom of Reach: 405.3 square miles

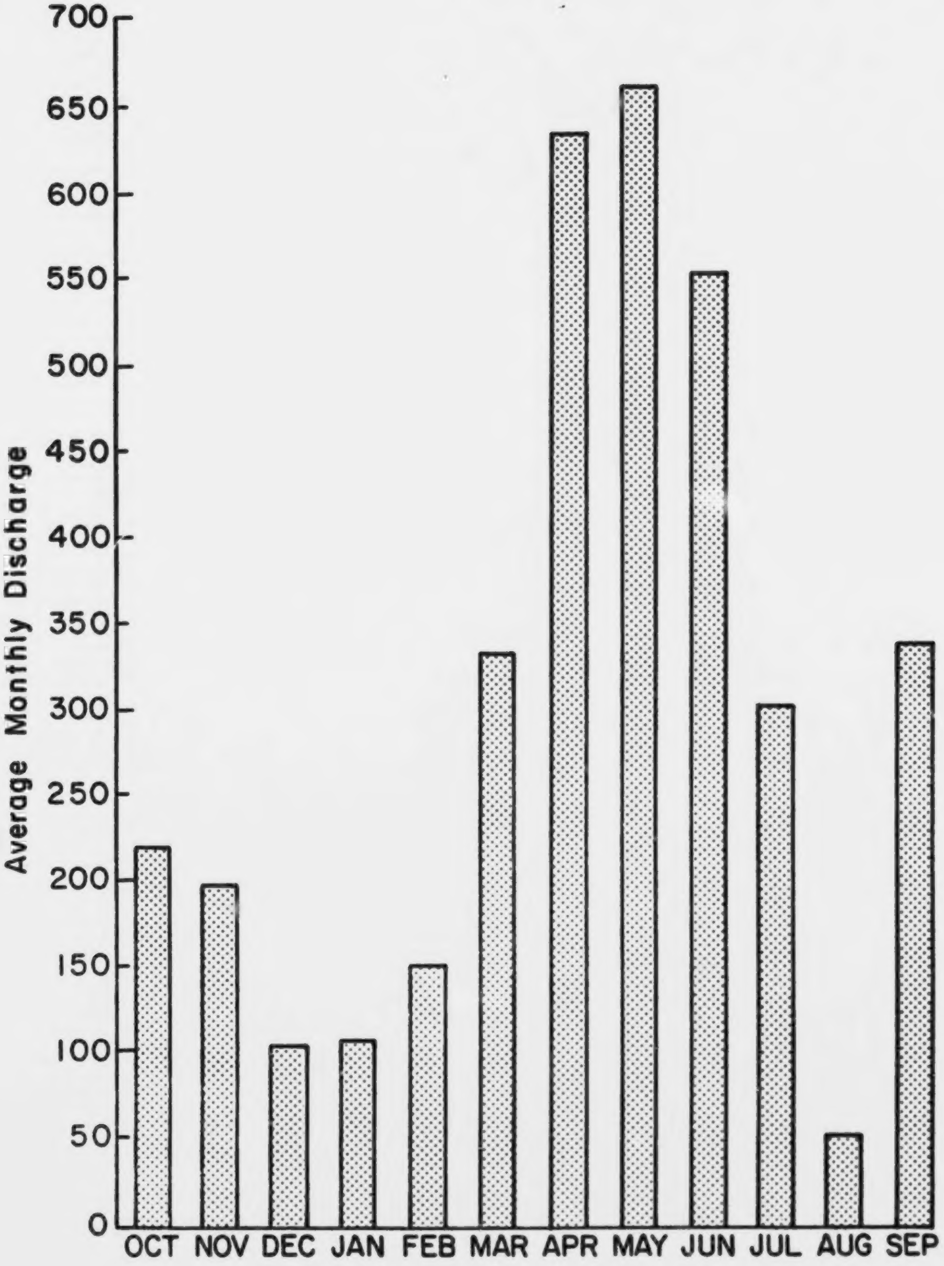
Gaging Station:

07-1698 Elk River at Elk Falls: 220 square miles
07-1700 Elk River near Elk City: 575 square miles

Included in the original data set were data from the U.S. Geological Survey report "Water Resources Data for Kansas." The data included were from water year 1978 and were on pages 15 and 297. Most of the data are from records of the U.S. Geological Survey.



Elk River at Elk Falls, Kansas (1968-1976)



Elk River near Elk City, Kansas (1939-1969)

ELK RIVER AT ELK FALLS. KANSAS
ARKANSAS RIVER BASIN
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1968	234.00	104.00	45.40	31.50	29.20	22.80	131.00	311.00	75.90	25.60	32.60	14.20	88.62
1969	197.00	390.00	143.00	84.00	158.00	332.00	444.00	614.00	758.00	34.20	2.29	169.00	276.23
1970	288.00	62.30	74.90	30.40	19.30	58.00	992.00	62.80	201.00	5.59	.25	13.90	149.87
1971	31.10	6.52	4.35	117.00	126.00	67.10	23.90	60.80	74.30	65.80	1.24	.33	47.86
1972	12.70	5.32	173.00	39.30	22.60	14.20	24.50	52.50	2.65	389.00	10.90	27.60	65.38
1973	19.40	398.00	229.00	394.00	265.00	1247.00	436.00	108.00	16.00	3.70	.77	128.00	270.73
1974	207.00	199.00	364.00	261.00	171.00	802.00	154.00	543.00	464.00	6.35	153.00	143.00	290.37
1975	389.00	930.00	215.00	309.00	426.00	477.00	140.00	354.00	508.00	20.00	12.70	11.70	314.31
1976	1.26	2.32	5.30	2.54	1.80	7.18	212.00	147.00	19.90	2080.00	5.73	2.76	210.48
AVERAGE	153.27	233.05	139.33	140.97	135.43	336.36	284.16	250.34	235.53	292.25	24.39	56.72	190.43
Q/QANN	6.831	10.052	6.210	6.283	5.501	14.992	12.256	11.158	10.159	13.025	1.087	2.446	100.000
COV VAR	.925	1.306	.859	1.016	1.039	1.299	1.078	.863	1.164	2.332	2.021	1.211	.549
SKEW	.327	1.733	.616	.880	1.163	1.384	1.776	.786	1.040	2.831	2.773	.899	-.269
MAXIMUM	389.00	930.00	364.00	394.00	426.00	1247.00	992.00	614.00	758.00	2080.00	153.00	169.00	314.31
MINIMUM	1.26	2.32	4.35	2.54	1.80	7.18	23.90	52.50	2.65	3.70	.25	.33	47.86

Monthly streamflow data.

STATISTICAL PARAMETERS FOR STATION 07169800 ELK RIVER AT ELK FALLS, KANSAS
ARKANSAS RIVER BASIN

	MONTH	ARTH. AVERAGE	LOG PARAMETERS MEAN	VARIANCE	STD. DEV.	SKEW
1	OCT	153.27	1.7793	.7025	.8331	-1.0480
2	NOV	233.05	1.7973	.8888	.9427	-.4516
3	DEC	139.33	1.8379	.5017	.7083	-1.0740
4	JAN	140.97	1.8306	.4686	.6845	-1.0222
5	FEB	135.43	1.7717	.5584	.7473	-.9952
6	MAR	336.36	2.0174	.6591	.8119	-.0464
7	APR	284.16	2.2031	.2982	.5461	-.4234
8	MAY	250.34	2.2297	.1782	.4221	.0952
9	JUNE	235.53	1.9147	.6731	.8204	-.5918
10	JULY	292.25	1.5653	.8158	.9032	.9755
11	AUG	24.39	.7039	.7483	.8651	.2077
12	SEPT	56.72	1.2505	.7817	.8842	-.7961
13	ANNUAL	190.43	2.1973	.0960	.3098	-.7273

SAMPLE SIZE 9 YEARS

LOG NORMAL DISTRIBUTION STATION 07169800 ELK RIVER AT ELK FALLS, KANSAS
ARKANSAS RIVER BASIN

YEARS FLOW IS NOT EXCEEDED

MONTH	9 IN 10	2 IN 3	1 IN 2	1 IN 5	1 IN 10	1 IN 20	Q2 - Q10
OCT	714.17	137.68	60.16	11.85	5.07	2.52	55.09
NOV	1013.60	159.11	62.70	10.08	3.88	1.76	58.82
DEC	557.13	138.60	68.85	17.44	8.51	4.71	60.34
JAN	510.61	133.11	67.69	17.96	8.97	5.06	58.72
FEB	536.63	123.67	59.11	13.88	6.51	3.49	52.60
MAR	1143.32	232.08	104.08	21.57	9.47	4.81	94.60
APR	800.31	273.79	159.64	55.37	31.84	20.17	127.79
MAY	590.00	257.52	169.72	74.87	48.82	34.31	120.90
JUNE	925.87	184.80	82.18	16.75	7.29	3.67	74.88
JULY	528.72	89.70	36.76	6.38	2.56	1.20	34.20
AUG	65.00	11.88	5.06	.95	.39	.19	4.66
SEPT	242.08	42.64	17.80	3.21	1.31	.63	16.49
ANNUAL	393.05	213.90	157.52	86.40	63.13	48.73	94.39

Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW

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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  07169800
                 ELK RIVER AT ELK FALLS, KANSAS
                     ARKANSAS RIVER BASIN

          DATA SKEW K FACTORS

 MONTH      SKEW                                                       RETURN PERIOD

                           10 YRS          6.67 YRS            2 YRS        1.25 YRS         1.11 YRS         1.05 YRS

OCT        -1.05          1.0961            .5234            .1878          -.7382          -1.3405          -1.9023
NOV         -.45          1.2083            .4813            .0907          -.8041          -1.3254          -1.7851
DEC        -1.07          1.1015            .5221            .1839          -.7416          -1.3407          -1.8982
JAN        -1.02          1.0907            .5248            .1917          -.7349          -1.3402          -1.9064
FEB        -1.00          1.1270            .5152            .1648          -.7574          -1.3400          -1.8776
MAR         -.05          1.2636            .4451            .0256          -.8328          -1.2968          -1.6875
APR         -.42          1.2037            .4836            .0953          -.8019          -1.3268          -1.7916
MAY          .10          1.2706            .4393            .0162          -.8363          -1.2915          -1.6717
JUNE        -.59          1.1986            .4863            .1004          -.7992          -1.3284          -1.79[illegible]
JULY         .98          1.3367            .3298           -.1359          -.8555          -1.1613          -1.37[illegible]4
AUG          .21          1.3003            .4080           -.0318          -.8497          -1.2589          -1.5863
SEPT        -.80          1.1653            .5015            .1326          -.7796          -1.3361          -1.8397
ANNUAL      -.73          1.1522            .5062            .1436          -.7720          -1.3382          -1.8528

FREQUENCY OF FLOWS

                              YEARS FLOW IS NOT EXCEEDED

           MONTH    9 IN 10     2 IN 3     1 IN 2    1 IN 5   1 IN 10   1 IN 20          Q2 - Q10

           OCT       498.85     165.21      86.44     14.47      4.53      1.53             81.91
           NOV       863.65     178.23      76.35     10.94      3.53      1.30             72.82
           DEC       415.09     161.32      92.93     20.54      7.73      3.11             85.20
           JAN       377.68     154.81      91.57     21.26      8.19      3.35             83.38
           FEB       411.00     143.43      78.49     16.06      5.89      2.34             72.59
           MAR      1104.59     239.17     109.17     21.94      9.22      4.44             99.96
           APR       725.30     293.26     179.95     58.24     30.10     16.78            149.86
           MAY       583.49     260.10     172.41     75.29     48.37     33.43            124.04
           JUNE      790.91     205.92      99.34     18.16      6.68      2.75             92.66
           JULY      592.46      72.98      27.71      6.20      3.28      2.09             24.42
           AUG        67.41      11.40       4.75       .93       .41       .21              4.33
           SEPT      190.87      49.42      23.32      3.64      1.17       .42             22.15
           ANNUAL    358.29     226.01     174.51     90.82     60.65     42.01            113.86

                  Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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ELK RIVER NEAR ELK CITY, KANSAS
ARKANSAS RIVER BASIN
UNITS OF DISCHARGE ARE CUSECS

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	ANNUAL
1939	2.00	9.40	5.70	8.10	9.80	35.50	260.00	350.00	195.00	6.00	1.80	.10	73.63
1940	0.00	0.00	0.00	0.00	.40	.70	55.30	101.00	74.30	1.30	28.70	27.80	24.14
1941	.90	118.00	55.00	296.00	253.00	69.20	1181.00	158.00	892.00	10.10	7.10	584.00	298.10
1942	1424.00	645.00	208.00	83.70	380.00	181.00	1293.00	268.00	1284.00	48.10	154.00	1564.00	623.37
1943	408.00	106.00	496.00	206.00	212.00	181.00	85.40	3692.00	1373.00	68.30	12.00	3.90	574.95
1944	14.10	2.70	13.10	11.40	43.40	1008.00	3400.00	785.00	186.00	23.50	45.90	226.00	478.02
1945	538.00	20.10	778.00	74.70	73.10	1368.00	2556.00	220.00	501.00	564.00	11.00	2740.00	785.05
1946	802.00	40.30	29.70	516.00	340.00	498.00	248.00	58.70	23.10	6.41	27.60	27.50	218.65
1947	1.67	150.00	46.90	24.60	11.90	403.00	3370.00	1374.00	275.00	54.10	7.89	6.19	475.42
1948	.74	.53	1.94	2.19	2.74	156.00	216.00	112.00	1330.00	3096.00	105.00	9.38	422.86
1949	2.91	102.00	10.80	770.00	1629.00	493.00	673.00	918.00	1575.00	417.00	51.40	182.00	560.00
1950	30.30	20.00	22.00	55.60	29.50	39.60	19.60	74.80	682.00	644.00	495.00	110.00	186.12
1951	11.80	7.14	8.87	9.30	70.60	62.90	220.00	1237.00	2721.00	2461.00	51.80	415.00	607.83
1952	68.30	603.00	164.00	210.00	164.00	850.00	585.00	95.50	49.30	6.57	3.17	.05	232.95
1953	0.00	0.00	1.14	1.04	76.00	18.90	4.20	96.40	2.14	3.24	0.00	0.00	16.65
1954	.60	13.40	6.16	.27	4.76	2.69	264.00	608.00	21.20	.62	9.73	1.35	78.31
1955	72.20	.02	.07	2.80	13.90	3.45	4.50	625.00	149.00	8.39	.80	0.00	74.22
1956	91.90	0.00	0.00	0.00	0.00	0.00	0.00	4.36	25.60	35.90	.05	0.00	13.32
1957	0.00	0.00	0.00	0.00	.83	1.83	156.00	1562.00	2322.00	68.40	34.00	40.20	348.35
1958	16.20	88.60	20.00	49.00	88.90	1885.00	756.00	514.00	75.50	440.00	19.70	259.00	353.58
1959	13.90	26.50	12.90	20.40	30.20	62.70	292.00	452.00	40.30	767.00	79.40	25.30	153.41
1960	1754.00	78.40	52.30	172.00	393.00	774.00	437.00	405.00	107.00	53.40	67.90	7.85	360.43
1961	29.50	13.80	79.10	10.10	54.70	312.00	721.00	4773.00	193.00	322.00	21.60	2874.00	787.69
1962	737.00	2085.00	462.00	481.00	202.00	286.00	85.80	26.30	86.20	84.30	3.49	497.00	418.37
1963	155.00	52.80	56.20	296.00	48.70	291.00	43.30	82.30	58.30	2.65	4.24	.19	91.78
1964	0.00	0.00	0.00	0.00	.38	.46	11.10	54.30	133.00	1.55	43.40	25.50	22.42
1965	.22	922.00	189.00	95.60	72.70	192.00	1603.00	130.00	550.00	31.60	9.48	189.00	328.70
1966	6.29	1.81	26.40	16.00	76.60	99.90	72.10	75.90	16.10	13.90	27.90	.02	36.08
1967	0.00	0.00	0.00	0.00	0.00	.01	18.60	19.60	518.00	372.00	23.10	469.00	117.79
1968	383.00	281.00	120.00	82.60	68.30	146.00	361.00	646.00	177.00	109.00	334.00	36.10	230.04
1969	292.00	802.00	402.00	209.00	358.00	813.00	737.00	1055.00	1628.00	60.90	7.13	248.00	549.14
AVERAGE	221.18	199.68	105.40	119.46	152.11	330.16	636.42	663.65	556.87	315.54	54.46	340.92	307.78
Q/QANN	6.099	5.329	2.906	3.294	3.823	9.104	16.983	18.300	14.861	8.701	1.502	9.098	100.000
COV VAR	1.937	2.152	1.752	1.543	1.974	1.370	1.455	1.589	1.311	2.206	1.912	2.133	.760
SKEW	2.501	3.283	2.377	2.130	4.238	1.972	2.114	2.902	1.591	3.277	3.367	2.897	.450
MAXIMUM	1754.00	2085.00	778.00	770.00	1629.00	1885.00	3400.00	4773.00	2721.00	3096.00	495.00	2874.00	787.69
MINIMUM	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.36	2.14	.62	0.00	0.00	13.32

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STATISTICAL PARAMETERS FOR STATION   07170000     ELK RIVER NEAR ELK CITY, KANSAS
  ARKANSAS RIVER BASIN


                                      ARTH.                  LOG PARAMETERS
                         MONTH       AVERAGE       MEAN    VARIANCE  STD. DEV.     SKEW


          1              OCT          221.18       .7633     3.9216     1.9803    -.8753
          2              NOV          199.68       .7421     4.2304     2.0568    -.9864
          3              DEC          105.40       .7743     3.5297     1.8787   -1.2300
          4              JAN          119.46       .8598     3.6232     1.9035   -1.2581
          5              FEB          152.11      1.3333     2.1019     1.4498   -1.8474
          6              MAR          330.16      1.6873     2.1792     1.4762   -1.7081
          7              APR          636.42      2.1550     1.5293     1.2367   -2.5448
          8              MAY          663.65      2.3898      .4654      .6822    -.3205
          9              JUNE         556.87      2.2826      .5596      .7481    -.4246
         10              JULY         315.54      1.6504      .9519      .9757     .0208
         11              AUG           54.46      1.0957     1.1884     1.0901   -2.0410
         12              SEPT         340.92      1.0676     3.6020     1.8979    -.9720
         13              ANNUAL       307.78      2.2802      .2703      .5199    -.8993


         SAMPLE SIZE   31 YEARS



LOG NORMAL DISTRIBUTION STATION   07170000     ELK RIVER NEAR ELK CITY, KANSAS
  ARKANSAS RIVER BASIN


                           YEARS FLOW IS NOT EXCEEDED

    MONTH    9 IN 10     2 IN 3     1 IN 2     1 IN 5    1 IN 10    1 IN 20            Q2 - Q10

   OCT       2004.55      41.00       5.80        .12        .02        .00                5.78
   NOV       2392.80      42.12       5.52        .10        .01        .00                5.51
   DEC       1523.61      38.05       5.95        .16        .02        .00                5.92
   JAN       1990.93      47.36       7.22        .18        .03        .01                7.20
   FEB       1555.64      90.21      21.54       1.30        .30        .09               21.24
   MAR       3800.41     209.23      48.68       2.78        .62        .18               48.05
   APR       5500.27     484.75     142.89      12.99       3.71       1.32              139.17
   MAY       1838.20     481.37     245.36      65.37      32.75      18.52              212.61
   JUNE      1744.31     401.35     191.69      44.95      21.07      11.27              170.63
   JULY       796.51     117.20      44.71       6.74       2.51       1.11               42.20
   AUG        311.38      36.59      12.47       1.51        .50        .20               11.97
   SEPT      3167.47      76.17      11.68        .29        .04        .01               11.64
   ANNUAL     884.43     318.58     190.63      69.58      41.09      26.61              149.54

             Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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LOG-PEARSON TYPE III DISTRIBUTION FOR STATION  07170000
                     ELK RIVER NEAR ELK CITY, KANSAS
                          ARKANSAS RIVER BASIN

         DATA SKEW K FACTORS

 MONTH       SKEW                                                        RETURN PERIOD

                           10 YRS          6.67 YRS              2 YRS            1.25 YRS            1.11 YRS            1.05 YRS

 OCT         -.88          1.1423            .5097               .1520             -.7663             -1.3392             -1.8627
 NOV         -.99          1.1252            .5157               .1662             -.7562             -1.3401             -1.8793
 DEC        -1.23          1.0479            .5333               .2205             -.7092             -1.3376             -1.9341
 JAN        -1.25          1.0544            .5321               .2163             -.7131             -1.3382             -1.9304
 FEB        -1.85           .9068            .5463               .3008             -.6175             -1.3058             -1.9927
 MAR        -1.71           .9220            .5459               .2930             -.6283             -1.3106             -1.9884
 APR        -2.54           .7343            .5332               .3724             -.4880             -1.2303             -2.0124
 MAY         -.32          1.2191            .4754               .0795             -.8096             -1.3218             -1.7691
 JUNE        -.42          1.2039            .4835               .0951             -.8020             -1.3268             -1.7914
 JULY         .02          1.2795            .4316               .0035             -.8408             -1.2841             -1.6508
 AUG        -2.04           .8542            .5457               .3255             -.5814             -1.2881             -2.0040
 SEPT        -.47          1.1221            .5165               .1685             -.7544             -1.3403             -1.8818
 ANNUAL      -.40          1.1469            .5081               .1481             -.7689             -1.3390             -1.8581

FREQUENCY OF FLOWS


                              YEARS FLOW IS NOT EXCEEDED

       MONTH    9 IN 10     2 IN 3     1 IN 2     1 IN 5    1 IN 10   1 IN 20           Q2 - Q10

      OCT       1060.16      59.25      11.59        .18        .01       .00              11.58
      NOV       1138.42      63.49      12.13        .15        .01       .00              12.12
      DEC        553.43      59.73      15.44        .28        .02       .00              15.42
      JAN        734.09      74.41      18.64        .32        .02       .00              18.62
      FEB        444.65     133.46      58.81       2.74        .28       .03              58.53
      MAR       1117.96     311.32     131.79       5.75        .57       .06             131.23
      APR       1156.30     652.23     412.61      35.61       4.30       .46             408.31
      MAY       1665.20     517.78     278.01      68.78      30.77     15.24             247.24
      JUNE      1524.83     440.88     225.80      48.16      19.50      8.76             206.30
      JULY       792.06     117.88      45.06       6.76       2.50      1.10              42.57
      AUG.       106.41      49.05      28.22       2.90        .49       .08              27.73
      SEPT      1574.91     111.66      24.40        .43        .03       .00              24.37
      ANNUAL     752.34     350.22     227.61      75.94      38.38     20.62             189.23

                  Q2 - Q10 IS THE 1 IN 2 YEAR FLOW MINUS THE 1 IN 10 YEAR FLOW
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ELK RIVER BED MATERIAL[1]

The following information on the bed material was prepared by the staff of the Kansas Office of the Soil Conservation Service. (Additional diagrams on sphericity and shape factor were included in the original data set. Also, the plotted size distribution curves were included.)

Cross Section 3-1: Inspected 4/February/80

The section was investigated with an 8 feet probe to determine the thickness of the material in the bottom. Five feet of water with 2-3 inches of ice was penetrated before the probe encountered the bottom. The probe was pushed another 3 feet without encountering any resistance. It is assumed the bottom consisted of alluvium because both banks consisted of alluvium.

Cross Section 3-3: Inspected 5/February/80

This section had 6.5 feet of water overlying the bottom. The probe was pushed another 12 inches before it become too hard to push. The material was not rock (Ls, SS, or Sh). It is assumed the bottom consisted of alluvium, because both banks consisted of alluvium.

Cross Section 3-4: Inspected 5/February/80

Rock was encountered 2.5 feet below the channel bottom. The rock is assumed to be sandstone since the Ireland Sandstone member of the Lawrence Shale formation is exposed in the bluff north of the channel. Rock is exposed not in either bank of the channel. The material is CL, ML type alluvium.

The armor layer is estimated to be 6 inches except for those areas where the 28 inches x 15 inches x 6 inches boulders are stacked one on another. The armor layer consists of sandstone fragments with the following gradation.

% Retained By Size	Particle Dimensions (inches)	
10	28 x 15 x 6	Sandstone
25	6 x 6 x 2	Sandstone
35	8 x 3 x 2	Sandstone
30	2 x 2.5 x 0.75	Sandstone

An alternate bar is located on the left (north) side of the channel. This bar contained 3 foot boulders along with materials found in the armor layer.

A sample of the bedload was not taken because of the depth of water (3 feet).

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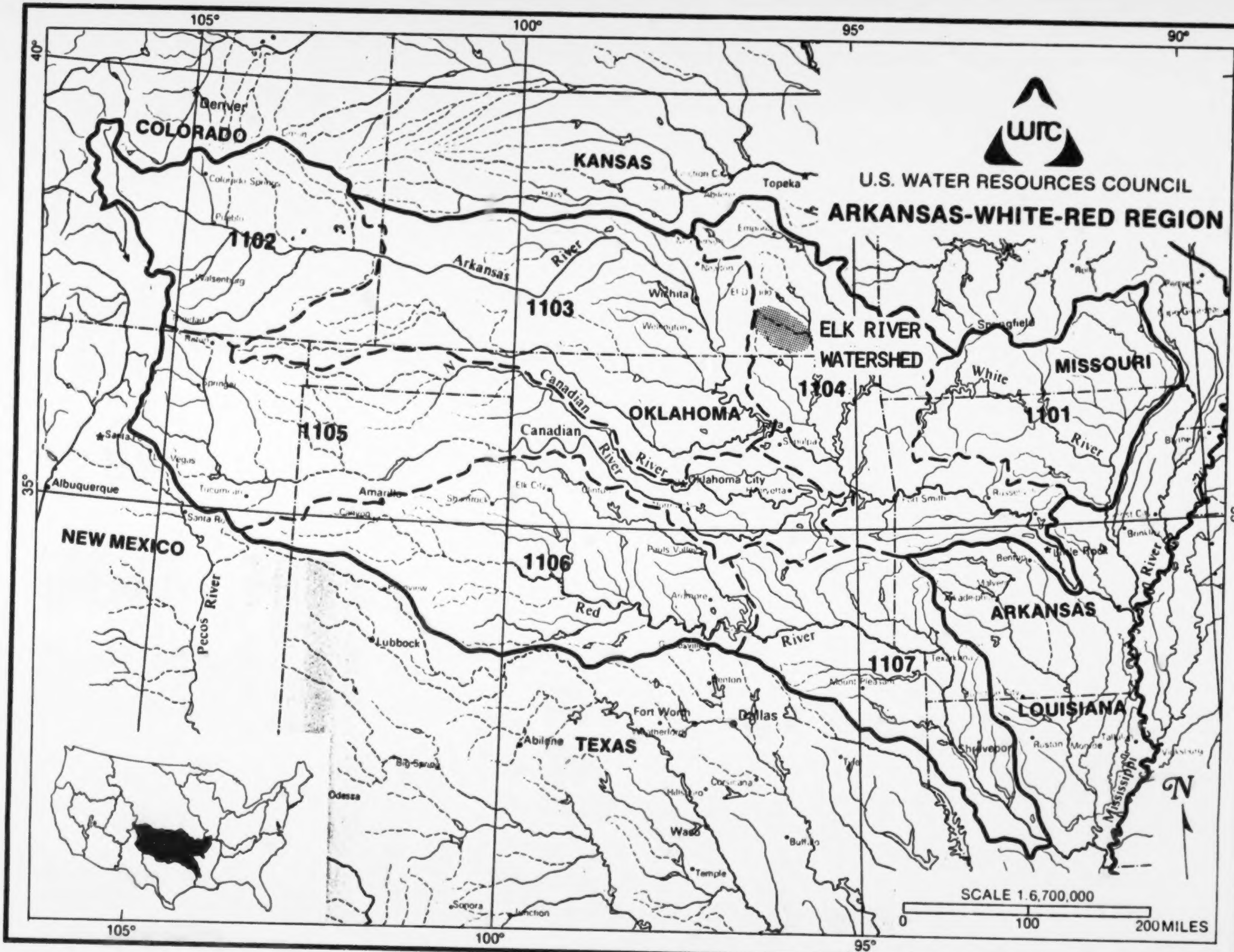
[1] Written by C. E. Deal, Soil Conservation Service, Kansas State Office



General location of Elk River, Kansas.

Cross Section 3-5: Inspected 5/February/80

Rock was encountered 6 feet below the bottom of the channel. The rock is assumed to be sandstone. The depth of water made it impossible to see the rock.

The Amazonia limestone member of the Lawrence Shale formation was exposed in the right bank. Alluvium was observed in the left bank.

The armor layer is estimated to range from 4-6 inches across the channel bottom. The armor layer consists of sandstone and limestone fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
1	28 x 17 x 14	Located by right bank limestone
2	24 x 15 x 2.5	Located by right bank limestone
10	16 x 10.5 x 2	Sandstone

Cross Section 3-5: Inspected 2/5/80

% Retained By Size	Particle Dimensions (inches)	
2	13 x 7 x 2	Sandstone
15	8.5 x 4 x 2	Limestone
30	5.5 x 3.5 x 2	Limestone
40	3.5 x 3.5 x 1	Limestone

A sample of the underlying material was taken and sent to the laboratory in Lincoln, Nebraska, for sieve analysis.

Cross Section 11-1: Inspected 5/February/80

The probe was pushed 18 inches below the bottom of the channel. The material pushed like shale. It is assumed to be shale of the Lawrence Shale formation. Alluvium was observed in both banks.

The armor layer is estimated at 4-6 inches. This layer consists of sandstone, limestone, and shale fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
15	43 x 38 x 1.75	Gray silty clay shale
5	40 x 26 x 11	Sandstone
10	28 x 20 x 4	Limestone
10	23 x 14 x 2	Limestone
5	13 x 11 x 1.5	Gray silty clay shale

% Retained By Size	Particle Dimensions (inches)	
20	9.5 x 6.5 x 1.5	Sandstone
20	6 x 4.5 x 1.5	Sandstone
10	4.5 x 2.5 x 3	Sandstone
5	4 x 3 x 1	Limestone

A sample of the underlying material was taken and sent to the laboratory in Lincoln, Nebraska, for sieve analysis.

Cross Section 11-7: Inspected 5/February/80

It was not possible to determine what is in the bottom at this section because the water depth is too deep (6-8 feet). Probing was done near the shore with the probe penetrating to 8 feet without encountering resistance. Alluvium was observed in both abutments. No rock was observed 1000 feet upstream or downstream from the bridge. Shale is assumed to underlie the bottom.

Cross Section 11-8: Inspected 6/February/80

Shale was encountered 4 feet below the bottom of the channel. The shale is the Lawrence Shale. The shale is a gray silty clay shale. Alluvium is in both banks of the channel.

The armor layer is estimated to range from 4-8 inches. This layer consists of limestone and sandstone fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
1	24 x 14 x 4	Sandstone
5	13 x 13 x 2	Sandstone
35	9 x 6 x 1	Limestone
5	7 1/2 x 12 x 4	Sandstone
40	7 x 5 x 1.75	Sandstone
14	4 x 3 x 2.5	Sandstone

A sample of the underlying material was taken and sent to the laboratory in Lincoln, Nebraska for sieve analysis.

Cross Section 11-9: Inspected 6/February/80

Under 6 inches of ice was 3.5 feet of water at this section. Probing indicates 6 to 8 feet of gravelly material on the bottom with softer material underneath to the depth probed (12-16 inches). Because of the water depth, samples were not taken. Alluvium was observed in both banks.

Between Cross Sections 11-11 and 11-12 at Station 2180: Inspected 6/February/80

Shale was encountered 12 inches below the bottom of the channel. The shale is the Lawrence Shale formation. The shale is fairly soft for 6 inches and then a very hard layer is encountered. This layer is assumed to be sandstone. The shale is a gray silty clay shale and is exposed 2 feet above the channel bottom in the right bank. Alluvium is observed in both banks. The right bank is vertical and eroding.

The armor layer is 4 inches thick and consists of limestone and sandstone fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
2	8 x 5.5 x 3	Sandstone
3	7 x 5 x 1	Sandstone
40	5 x 3 x .5	Shaley Limestone
4	4.5 x 3 x 2.5	Limestone
25	3.5 x 2.5 x .5	Sandstone
26	3 x 2 x .25	Sandstone

A sample of the underlying material was taken and sent to the laboratory in Lincoln, Nebraska for sieve analysis. A soil sample was taken from the right bank.

Cross Section 11-12: Inspected 6/February/80

Shale is exposed in the bottom of the channel. A few 2 feet sandstone fragments are scattered over the bottom. No armor or bedload exists at this location. Depth of weathering in the shale is 8 inches. The shale is the Lawrence Shale formation. Alluvium is exposed in both banks. The shale is exposed 12 inches above the waterline in the left abutment. The shale is a gray silty clay shale. The left bank is vertical and eroding.

Between Cross Sections 11-14 and 11-15 at Station 2080: Inspected 6/February/80

Shale was encountered 3.5 feet below the bottom of the channel. This shale is the Lawrence Shale formation. It is a gray silty clay shale.

The armor layer is 4-6 inches. The layer consists of limestone and sandstone fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
1	11.5 x 6 x 3	Limestone
5	8 x 6 x 1	Limestone
25	5 x 3 x 2	Limestone

% Retained By Size	Particle Dimesnions (inches)	
30	4 x 3 x .75	Limestone
19	2.5 x 2.25 x .25	Limestone
20	2.5 x 2 x .75	Limestone

Alluvium is exposed in both abutments. A sample of the underlying was taken and will be sent to the Soil Mechanic Laboratory in Lincoln, Nebraska for sieve analysis.

Cross Section 11-15: Inspected 6/February/80

Rock was encountered 12 inches below the bottom of the channel. This rock is interpreted to be limestone.

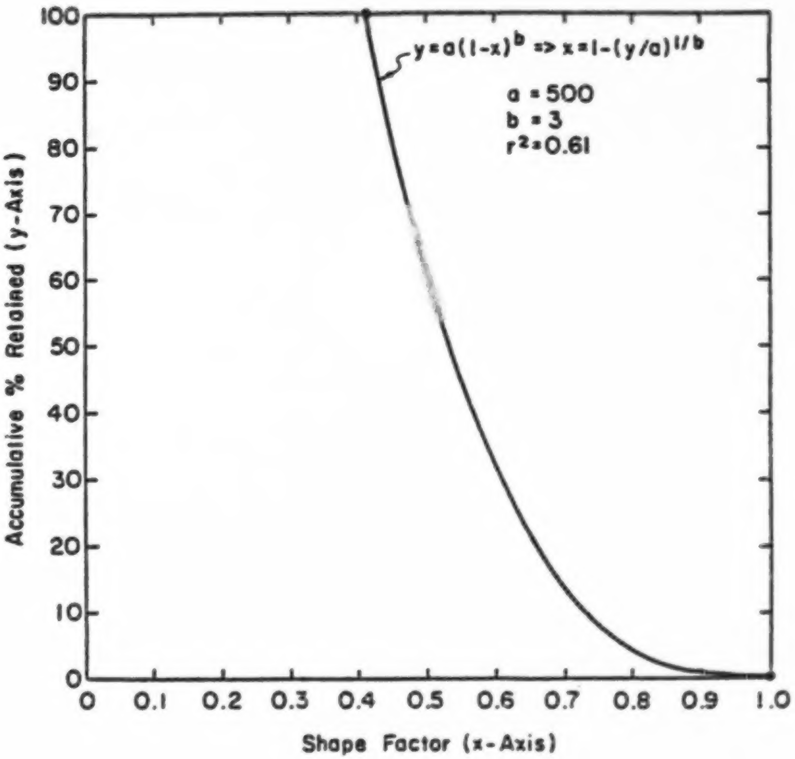
The armor thickness is 6 inches with 6 inches of bedload underlying the armor. The armor layer consists of limestone and sandstone fragments with the following gradation:

% Retained By Size	Particle Dimensions (inches)	
5	22 x 17 x 4	Limestone
5	16 x 11 x 3	Shaley Sandstone
15	11.5 x 7 x 4	Limestone
15	9 x 7 x 3	Limestone
10	8 x 5 x 1.5	Limestone
10	3.5 x 2.5 x 2	Sandstone
5	3 x 2.5 x 1	Sandstone
25	3 x 2.25 x .25	Limestone
10	1.5 x 1.75 x .5	Limestone

Alluvium exposed in both banks of the channel.

Cross Section 13-2: Inspected 6/February/80

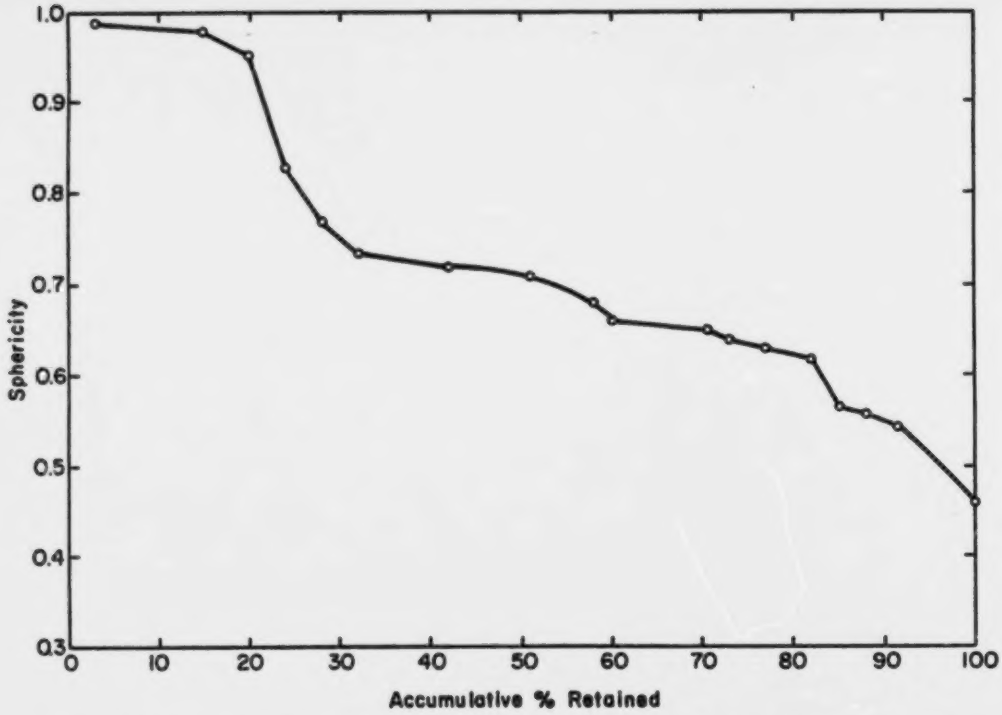
Limestone is exposed in the bottom of the channel. The limestone is clear of any armor or bedload. This limestone is the Amazonia limestone member of the Lawrence Shale formation.



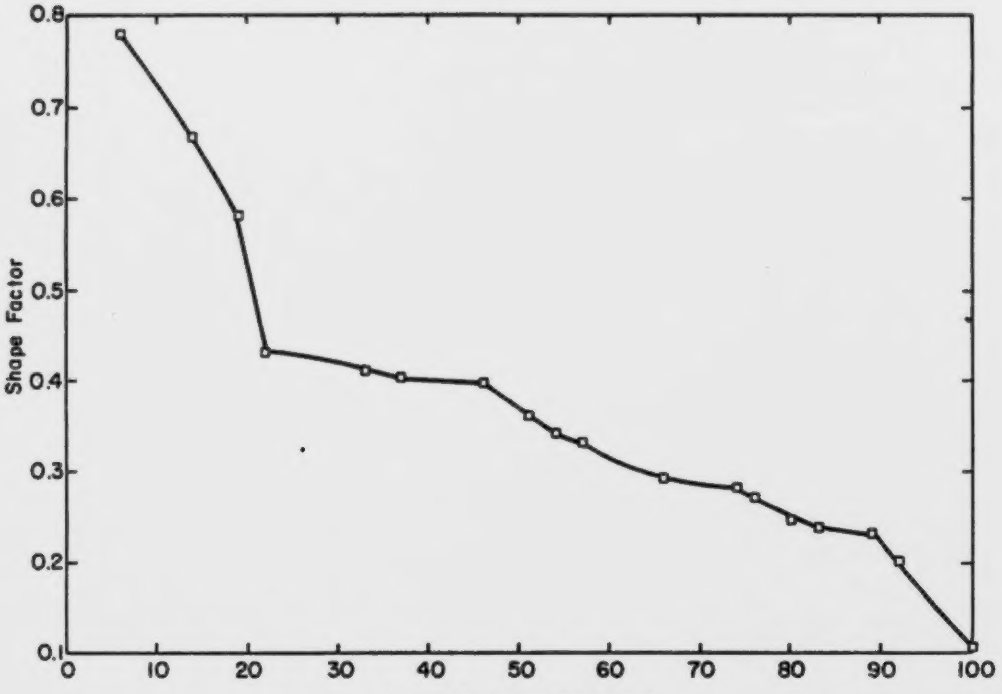
Channel bed material armor, shape factor- composite curve fit. $S_f = [(a \cdot b \cdot c)^{1/3}]/a$ where a is the longest axis of rock particle and b and c are the remaining axis of rock particle.

LOCATION	MECHANICAL ANALYSIS - PERCENT FINER												
	6"	3"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 10	No. 40	No. 60	No. 200
Stream, x-sect 3-5, Bedload	8	9	13	14	17	24	34	45	53	64	71	81	100
Stream, x-sect 11-1 Bedload	7	7	8	9	11	19	32	48	57	70	78	87	100
Stream, x-sect 11-8, Bedload	4	5	6	7	9	17	28	39	45	56	65	84	100
Stream, x-sect 11-11, Bedload	8	9	12	13	17	28	43	59	68	80	89	98	100
Stream, x-sect 11-15, Bedload	7	8	10	12	16	24	36	53	64	82	90	98	100

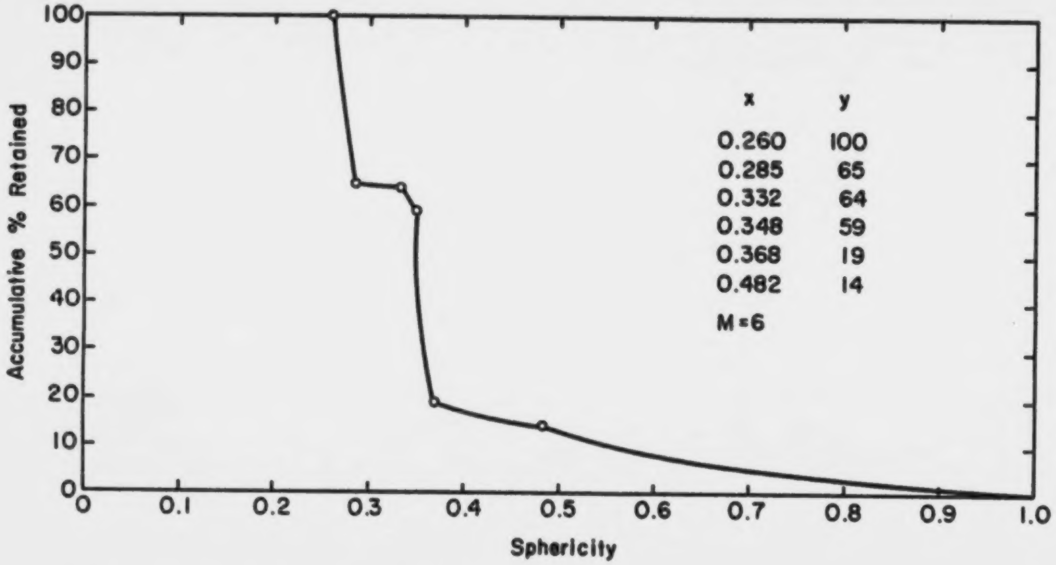
Soil mechanics laboratory data (lower Elk River stream channel).



Sphericity compared with percent retained (Elk River Watershed, Kansas; cross section 11-8, 3/31/80 C.T.)



Shape factor compared with percent retained (Elk River Watershed, Kansas; cross section 11-8, 3/31/80 C.T.)

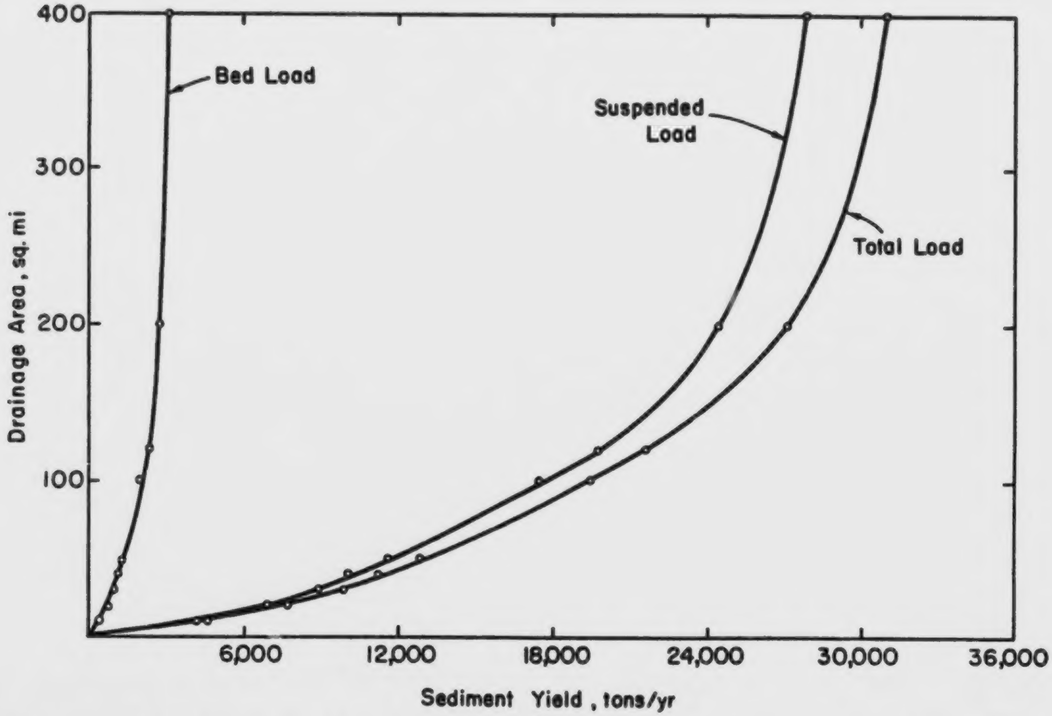


x	y
0.260	100
0.285	65
0.332	64
0.348	59
0.368	19
0.482	14

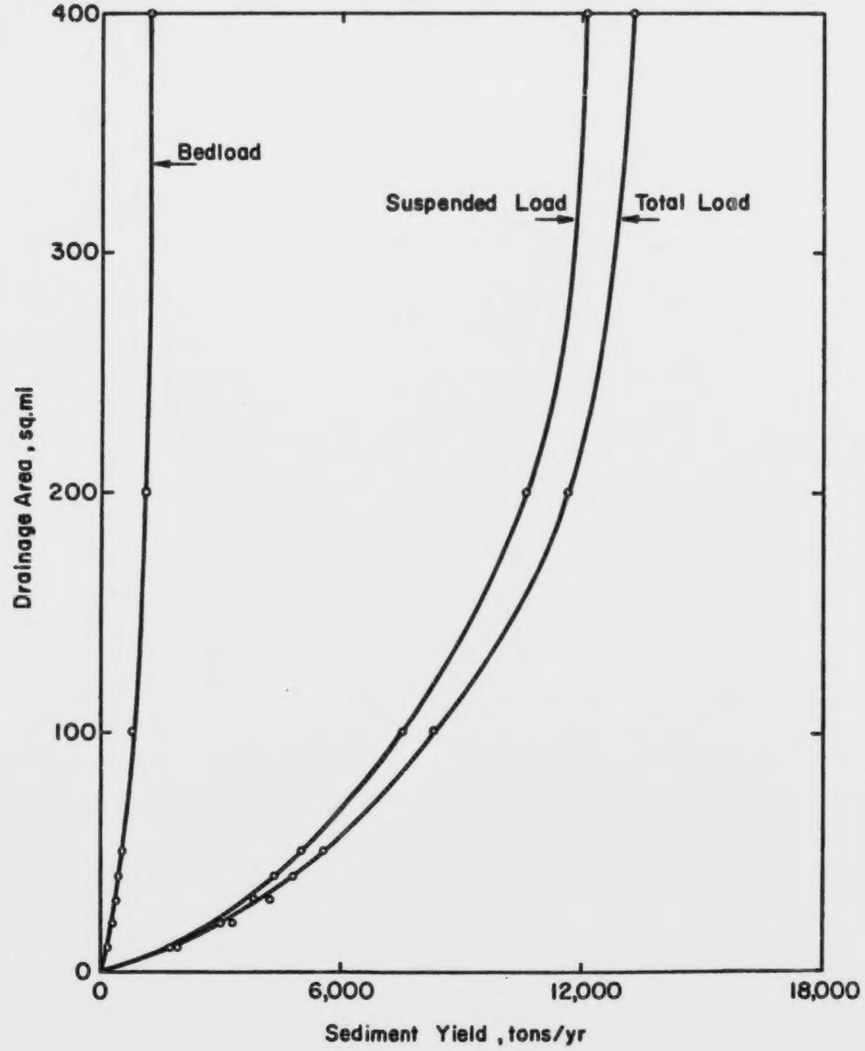
Channel stability-armor (Elk River, Kansas; cross section 11-8, 2/6/80 C.E.D.)

ELK RIVER SEDIMENT YIELD

The following information on sediment yield was developed by the staff of the Kansas Office of the Soil Conservation Service. Only one suspended sediment measurement has been made and is included.



Drainage area compared with sediment yield without project (Elk River, Kansas 1980).

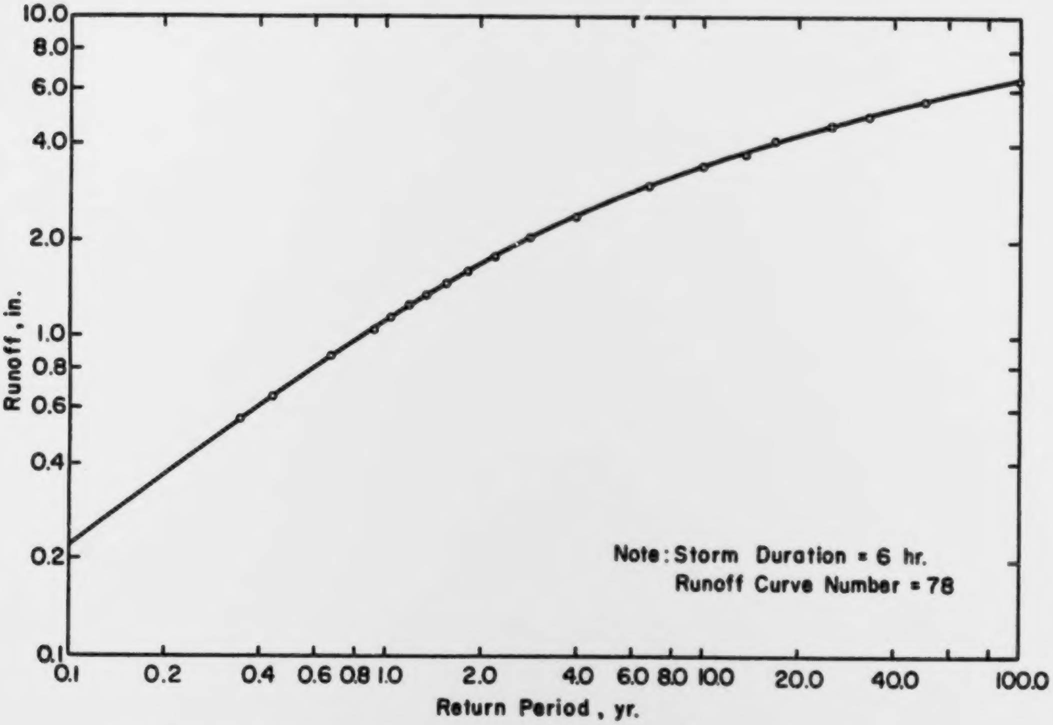


Drainage area compared with sediment yield with project (Elk River, Kansas 1980).

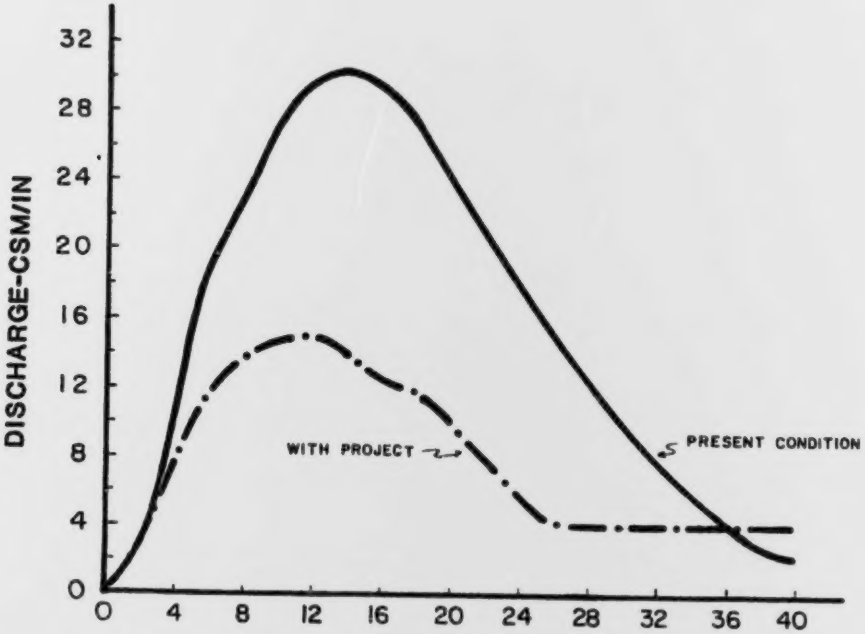
Suspended Sediment Measurement

07169800-Elk River at Elk Falls, Kansas

Date	Time	Discharge (cfs)	Specific Conductance (micromhas)	Sediment Suspended (MG/L)	Sediment Discharge (T/Day)
11-15-77 (1978 water year)	1430	154	460	61	25



Frequency-runoff curve (annual series) (Elk River, Kansas).



Unit discharge hydrograph (Elk River, Kansas).

Unit Discharge Hydrograph Table (Elk River, Kansas)

hr. Time	Discharge-csm/in.* condition Present	Discharge-csm/in.* condition with Project**
0	0	0
2	3.69	2.69
4	11.32	8.21
6	18.51	11.92
8	23.02	13.93
10	26.66	14.61
12	29.55	14.74
14	30.29	13.93
16	29.41	12.22
18	27.21	10.91
20	24.23	9.36
22	21.01	7.88
24	17.85	5.45
26	14.87	4.00
28	12.12	
30	9.72	
32	7.55	
34	5.56	
36	3.86	
38	2.70	
40	2.00	

Reach	Drainage Area Sq. Mi.
11 U.S.	284.5
11 D.S.	297.1
3 U.S.	341.0
3 D.S.	405.3

U.S. - Upstream

D.S. - Downstream

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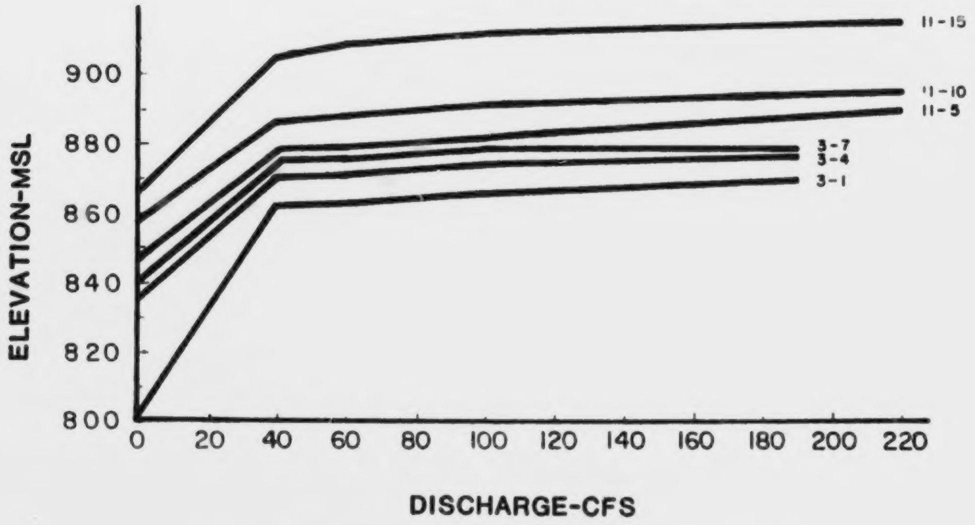
*discharge in cfs per sq. mi. per inch of runoff applicable for reaches 3 and 11.

**drainage area controlled equal 239.83 sq. mi. with average uncontrolled release rate from structures equal to 20 csm.



Elevation-discharge plots (Elk River, Kansas).

Elevation-Discharge Table (Elk River, Kansas)*

				Bankfull				Out-of-Bank Flow									
		Q=0		Q		ft.^{2}	fps	40 csm		60 csm		100 csm		190 csm		220 csm	
x-sec	DA	Elev-msl	E-msl	cfs	csm	A	$\bar{v}$	Q-cfs	E-msl	Q-cfs	E-msl	Q-cfs	E-msl	Q-cfs	E-msl	Q-cfs	E-msl
Reach 3																	
3-1	405.3	832.2	855.5	6500	16	1862	3.49	15,708	863.6	23,562	865.8	40,530	865.6	77,007	870.5	—	
3-2	392.2	832.2	855.0	3750	10	2086	1.80	15,688	866.5	23,532	867.4	39,220	868.9	74,518	871.8	—	
3-3	390.5	833.2	864.5	9200	24	3586	2.57	15,620	867.9	23,430	869.0	39,050	870.9	74,195	873.6	—	
3-4	388.0	835.9	866.0	9700	25	3123	3.11	15,520	869.5	23,280	871.1	38,800	873.1	73,720	875.9	—	
3-5 **	387.6/341.3	837.2	867.5	7250	21	2180	3.33	13,652	871.9	20,478	873.4	38,760	875.5	73,644	878.1	—	
3-7	341.0	840.0	869.5	7250	21	3232	2.24	13,640	874.0	20,460	875.3	34,100	877.3	64,790	879.9	—	
Reach 11																	
11-1	297.1	842.5	873.0	8750	29	4630	1.89	11,884	874.9	17,826	876.5	29,710	879.6	—		65,362	882.4
11-3	297.1	842.5	871.0	6300	21	3078	2.05	11,884	875.1	17,826	876.8	29,710	880.6	—		65,362	887.9
11-4	296.6	844.8	871.0	5500	19	2261	2.43	11,864	876.6	17,796	878.7	29,667	881.7	—		65,252	888.6
11-5	296.3	845.5	872.5	6500	22	1802	3.61	11,852	877.7	17,778	879.8	29,630	882.4	—		65,186	888.9
11-7	295.2	849.2	877.5	8300	28	2667	3.11	11,808	880.0	17,712	881.9	29,520	884.3	—		64,944	889.8
11-8	294.0	856.0	881.5	9600	33	3924	2.45	11,760	883.3	17,640	885.5	29,400	887.5	—		64,680	891.5
11-9	291.8	857.2	881.0	6750	23	2091	3.23	11,672	885.8	17,508	888.1	29,180	890.5	—		64,196	893.8
11-10	291.2	858.5	880.0	5000	17	1999	2.50	11,648	887.2	17,472	889.6	29,120	892.4	—		64,064	895.9
11-11	290.8	860.6	886.0	7000	24	2730	2.56	11,632	889.3	17,448	890.8	29,080	893.6	—		63,976	897.2
11-12	288.4	861.0	869.0	8500	29	2074	4.10	11,536	891.2	17,304	893.1	28,840	895.5	—		63,448	899.0
11-14	286.3	863.2	891.0	8500	30	3546	2.40	11,452	893.2	17,178	895.6	28,630	897.8	—		62,986	901.2
11-15	285.7	865.6	895.0	10,750	38	3030	3.88	11,428	895.5	17,142	898.2	28,570	901.0	—		62,854	904.7

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*Spatially-varied steady flow assumption was used.

**Contributing drainage area changes at low flows due to confluence.

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This workshop assembled 20 of the world's most qualified individuals to discuss solutions to downstream problems stemming from hypothetical impoundments or diversions, or both, on three selected Western United States rivers.

The proceedings of the workshop are composed of results prepared by the 20 participants along with reporters' comments and workshop summaries. These reports describe how the author perceives the problem of river response and determines a solution to a set of hypothetical man-induced changes.

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